STATE PROJECT NO. 95-522

BRIDGE No. 0163, I-95 OVER WEST RIVER

NEW HAVEN / WEST HAVEN, CT

HYDRAULIC AND SCOUR ANALYSIS REPORT FOR FINAL BRIDGE DESIGN

August 2003

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Submitted to:
Connecticut Department of Transportation
Newington, Connecticut

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EXECUTIVE SUMMARY

Parsons, Brinckerhoff, Quade & Douglas Inc. has prepared the final design of Bridge No. 0163, I-95 over the West River for the Connecticut State Department of Transportation. ConnDOT is upgrading the bridge as part of improvements to Interstate-95 in the New Haven area. The proposed design calls for replacement of the existing bridge with five piers of similar configuration to carry a widened bridge deck.

A hydraulic and scour analysis report on the preliminary design alternatives for the I-95 bridge over West River was submitted to ConnDOT in the summer of 2001. The purpose of this report is to present the hydraulic and scour analysis for the final design of the I-95 bridge over West River, as well as for the worst-case temporary conditions during construction. The analysis consists of a hydrologic analysis, hydraulic modeling using two-dimensional finite element modeling, and a scour analysis.

A summary of the existing and proposed bridge characteristics is presented below.

LOCATION:
Structure No.: 0163 Project No.: 95-522
Town: New Haven / West Haven
Highway: I-95
Watercourse: West River

EXISTING STRUCTURE:
Superstructure Type: Multi-Steel Girder.
Substructure Type: Piers 2-7, Plinth and Pier Bent; Piers 8-10, Pier Bent.
Foundation Type: Piers 2-7, Cast-in-Place Concrete Pile Supported Footer; Piers 8-10, Steel H-Pile Supported Footer.

PROPOSED STRUCTURE:
Superstructure Type: Multi-Steel Girder.
Substructure Type: Piers 1, 4 and 5, Pier Bent; Piers 2 and 3, Plinth and Pier Bent.
Foundation Type: Pile Supported Footer.
NBIS Item 113 – Scour Critical Bridges: 5
NBIS Item 71 – Waterway Adequacy: 9
NBIS Item 61 – Channel and Channel Protection: 7-8
Scour Risk Designation: Low Risk

Estimates of scour depth for the proposed bridge are summarized in the following table, along with estimates for the existing bridge during worst-case construction conditions.

SUMMARY OF SCOUR DEPTH ESTIMATES					
AT THE PROPOSED I-95 CROSSING OVER WEST RIVER					
Condition	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5
100-year Tidal Storm Surge	0.9m	6.1m	2.6m	1.7m	0.2m
500-year Tidal Storm Surge	1.4m	7.1m	3.0m	2.4m	0.9m

25-Year Construction Conditions	0.8m	7.3m	4.6m	1.4m	0.1m
AT THE EXISTING I-95 CROSSING OVER WEST RIVER					
25-Year Construction Stage	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
Stage 2	6.6m	8.0m	9.3m	5.4m	5.2m

Based on the hydraulic and scour analyses the proposed structure has been designed to be structurally stable for the analyzed storm events. The highest scour potential occurs at the existing piers due to the obstruction caused by the cofferdams and temporary sheet piling enclosures that will be placed during construction. Up to 9.3 meters of scour are estimated at Existing Pier 5. Since traffic will be maintained on the existing bridge during construction, careful analysis and monitoring of the existing piers is recommended.

INTRODUCTION

Final design of Bridge No. 0163, I-95 over West River between New Haven and West Haven, Connecticut, is being prepared by Parsons Brinckerhoff Quade and Douglas, Inc. (PBQD) for the Connecticut Department of Transportation (ConnDOT). Hydraulic and scour analyses are prepared as part of the final design.

The location of the bridge is shown in Figure 1. The existing bridge is a 12 span structure with a length of 348m and a width of 28m. ConnDOT is upgrading the bridge as part of improvements to Interstate-95 in the New Haven area. The proposed design calls for replacement of the existing bridge with five piers of similar configuration to carry a widened bridge deck.

A hydraulic and scour analysis report on the preliminary design alternatives for the I-95 bridge over West River was submitted to ConnDOT in the summer of 2001. The purpose of this report is to present the hydraulic and scour analysis for the final design of the I-95 bridge over West River, as well as for the worst-case temporary conditions during construction. The analysis consists of a hydrologic analysis, hydraulic modeling using two-dimensional finite element modeling, and a scour analysis. The hydrologic and hydraulic analysis procedures are the same as used for the preliminary analysis. The scour analysis procedure has been updated to reflect revisions to the methodology for estimating scour at complex piers that were made since the time of the preliminary analysis. The revisions to the scour analysis procedure have been made based on the fourth and latest edition of the Federal Highway Administration's Hydraulic Engineering Circular No. 18, *Evaluating Scour at Bridges* (HEC-18) published in May of 2001.

The existing I-95 structure spanning the West River has no documented history of scour related problems. The underwater inspection report for Bridge No. 0163 by Lan-Robinson Associates, Inc. dated March 1995 documents the channel bed in the vicinity of the I-95 bridge as consisting of silt and shells with no evidence of undermining or scour at the bridge piers. Based on the diver's observations the bridge was given a scour susceptibility rating of eight. In July of 1996 Close, Jensen and Miller submitted to ConnDOT a Scour Assessment Report for Bridge No. 0163. In the scour assessment report Close, Jensen and Miller, P.C. noted that the I-95 crossing of the West River has shown no history of scour and gave the bridge a NBIS Item 113 rating of eight.

Estimates of scour depth for the proposed bridge are summarized in the following table, along with estimates for the existing bridge during worst-case construction conditions.

SUMMARY OF SCOUR DEPTH ESTIMATES					
AT THE PROPOSED I-95 CROSSING OVER WEST RIVER					
Condition	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5
100-year Tidal Storm Surge	0.9m	6.1m	2.6m	1.7m	0.2m
500-year Tidal Storm Surge	1.4m	7.1m	3.0m	2.4m	0.9m
25-Year Construction Conditions	0.8m	7.3m	4.6m	1.4m	0.1m

AT THE EXISTING I-95 CROSSING OVER WEST RIVER					
25-Year Construction Stage	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
Stage 2	6.6m	8.0m	9.3m	5.4m	5.2m

The highest scour potential occurs at the existing piers due to the obstruction caused by the cofferdams and temporary sheet piling enclosures that will be placed during construction. Up to 9.3 meters of scour are estimated at Existing Pier 5. Since traffic will be maintained on the existing bridge during construction, careful analysis and monitoring of the existing piers is recommended.

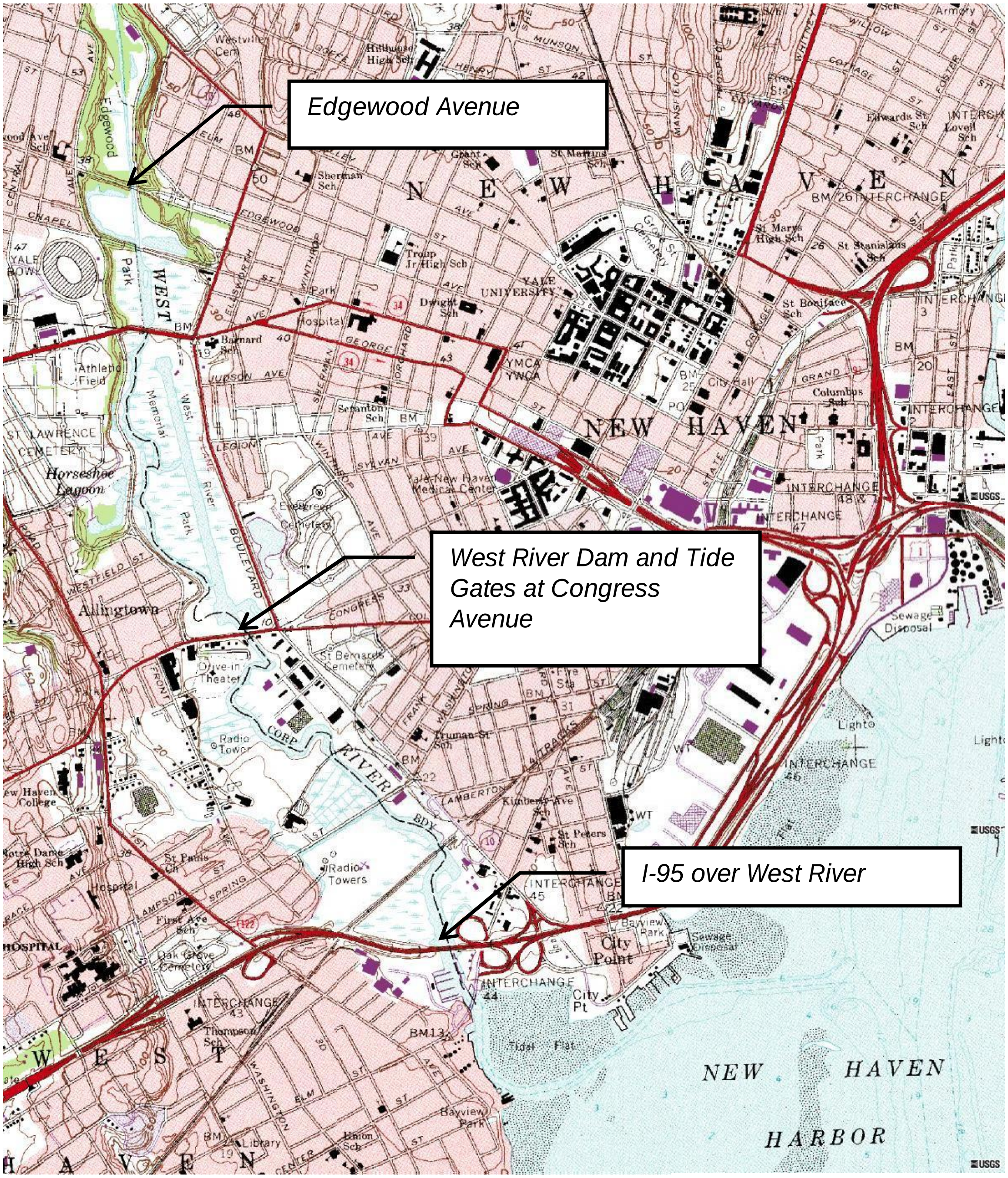

Figure 1: Vicinity Map

HYDROLOGIC ANALYSIS

OBJECTIVE

The purpose of the hydrologic analysis is to provide estimates of riverine discharges and tidal stages for the scour design and scour check events. The scour design event is the 100-year frequency event and the scour check event is the 500-year frequency event.

SITE DESCRIPTION

The West River flows for approximately 26.5km (16.5miles) from the northern edge of Bethany, Connecticut, along the western side of the Naugatuck State Forest to New Haven Harbor, passing through the towns of Woodbridge, West Haven, and New Haven. Tributaries to the West River also flow through Hamden. The lower 4.6km (2.9 miles) of the West River form the boundary between New Haven and West Haven. The I-95 crossing of the West River is located approximately 305m (1000 feet) upstream of the river's confluence with New Haven Harbor. New Haven Harbor is a large open bay on Long Island Sound. The primary inflow to the harbor is the Quinnipiac River.

Mapping and site data used in assessing the hydrology of the West River was obtained from the website of the University of Connecticut Map and Geographic Information Center (MAGIC) in the form of geospatial vector data. This data includes geo-referenced topographic mapping, land use, soil type, surficial material classification, hydrography, drainage basin delineation, and FEMA floodplain delineation on a town-by-town basis. The data is in Connecticut State Plane coordinates, NAD 27. All data except for the FEMA floodplain data is also available on a quad-by-quad basis.

The USGS quadrangles surrounding West River include New Haven, Ansonia, Naugatuck, and Mount Carmel. The majority of the area draining to the West River lies within the New Haven and Mount Carmel quads. The drainage area is approximately 9117 ha, (35.2 square miles), comprised of 29 subbasins of the West River, Sargent River, and Wintergreen Brook subregions of the South Central Western Complex, which lies in the South Central Coast Drainage Basin. Drainage basin areas are taken from the delineation performed by the Connecticut Department of the Environment on the 7.5 minute USGS quad maps. This data is included in the Drainage Basins Data Layer available from MAGIC (See Figure 2).

Deciduous forest and pasture cover most of the upper reaches of the basin, and more than half of the middle section. The lower reaches are more densely populated, with approximately 2130ha (8.2 square miles) of medium-density residential, commercial, and impervious areas. There are numerous ponds, lakes, and marshes throughout the drainage basin, including Konolds Pond, Lake Wintergreen, Lake Dawson, Glenn Lake, Lake Chamberlain, Lake Bethany, and Lake Watrous. These areas, constituting approximately 6% of the surface area of the basin, provide significant storage for riverine flooding. Storage in the basin is further increased by control structures including the Lily Pond Dam, the West River Dam, and tide gates at Congress Avenue.

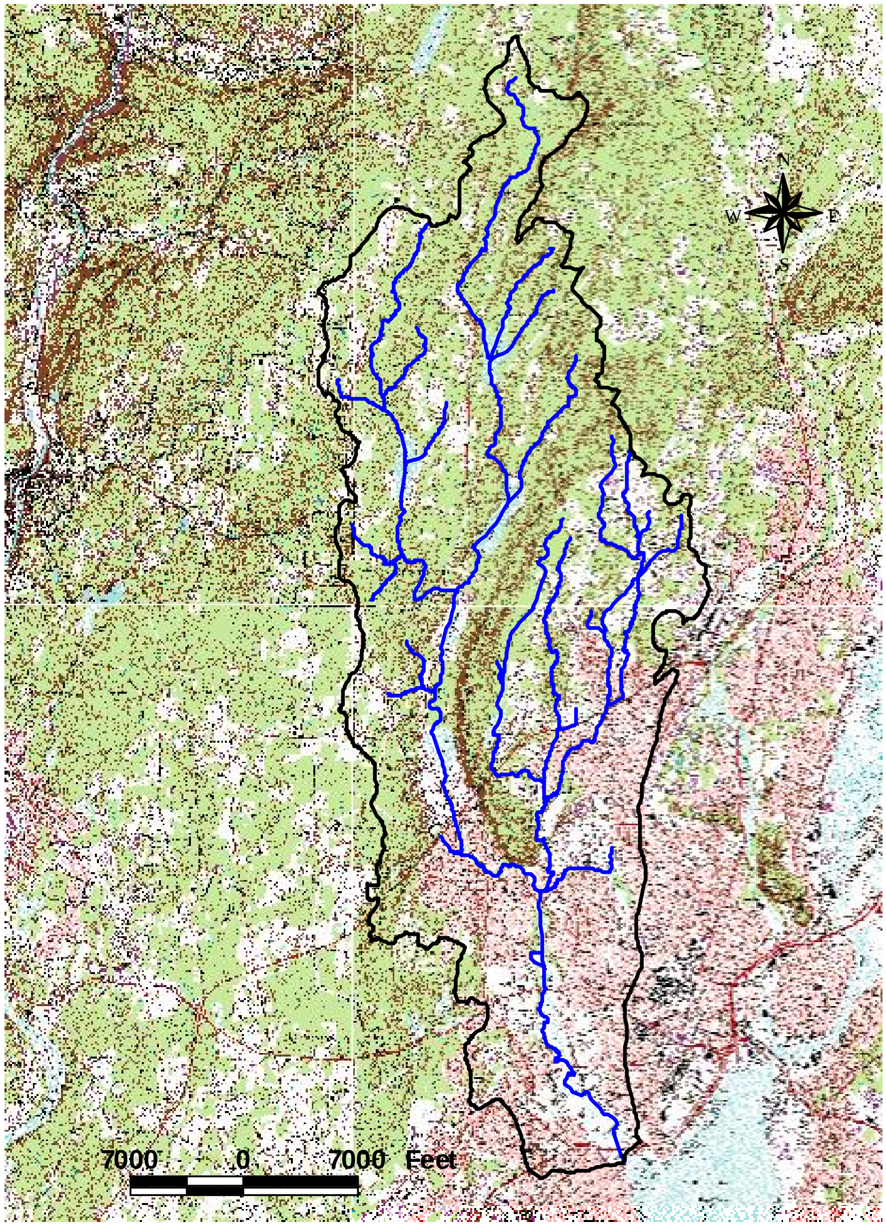

Figure 2: Drainage Area and Subarea Delineation

PREVIOUS FLOODPLAIN STUDIES

The Federal Emergency Management Agency (FEMA) prepared Flood Insurance Studies for the cities of New Haven and West Haven in 1980. The Flood Insurance Studies were supplemented with wave height analyses in 1982. The FEMA studies classify the West River as being subjected to tidal influence from New Haven Harbor upstream to Edgewood Avenue, 5km (3 miles) upstream of the I-95 crossing.

The flows used in the FEMA study for the downstream limits of the riverine portion of the West River are listed in Table 1. Note that FEMA's peak flood elevations for the tidal portion of the West River, including the I-95 crossing, are based on tidal storm surges.

TABLE 1: PEAK DISCHARGES FOR WEST RIVER FEMA FLOODPLAIN ANALYSIS		
Frequency	**Flow**	
	(cfs)	**m^3/sec**
10-year	2750	78
50-year	4000	113
100-year	4800	136
500-year	6300	178

HYDROLOGY FOR SCOUR ANALYSIS

The hydraulic analysis of the I-95 crossing requires the determination of peak flows or tidal stages at the structure. Because the drainage area conveyed by the I-95 crossing of the West River is greater than 1 square mile, the design frequency is the 100-year frequency. The 500-year frequency is also evaluated. This is in accordance with the procedures outlined in Appendix 6.A of the ConnDOT Drainage Manual.

Due to the proximity of the crossing to New Haven Harbor, and the storage characteristics upstream of the crossing, riverine floods are dampened in magnitude at the structure, but the structure is directly exposed to tidal storm surges. As the West River approaches New Haven Harbor it passes through a series of large pools with restrictive bridges and tide gates that dissipate the magnitude of upland flooding. Using HEC-18 guidelines, the crossing is classified as a tidally controlled crossing. Specific characteristics that support classification as a tidally controlled crossing include:

- The FEMA FIS for the City of New Haven locates the upstream limit of tidal influence due to the 100-year storm surge at Edgewood Avenue, which is roughly 5km (3 miles) upstream of the I-95 crossing. The FIS reports that the majority of flooding in New Haven is caused by coastal storms.
- The West River Dam and Tide Gates at Congress Avenue, just downstream of the Columbus Avenue / Orange Avenue bridge crossing, are located approximately 2.4km (1.5 miles) upstream of the I-95 crossing (See Figure 1).
- The Amtrak Rail Bridge and the Spring Street Bridge restrict flow, effectively creating two large detention areas between the tide gates and I-95 that further dampen riverine flood peaks.

- During a flood event, flood peaks would be dampened by upstream storage and by the tide gates, and would not influence the impact of the tidal storm surge on the structure. Moreover, storage and routing through the drainage basin creates a considerable lag in the flood hydrograph such that the peak riverine flows would not coincide with the tidal surge in the vicinity of the crossing.
- The I-95 crossing is located directly upstream of the Kimberly Bridge, without any intervening storage, and the Kimberly Bridge is directly exposed to storm surges on New Haven Harbor.

Because the crossing is tidally controlled, or at least tidally influenced, a storm surge hydrograph (stage graph) is used for hydraulic modeling. The 100-year and 500-year tidal storm surge hydrographs for New Haven Harbor have been developed based on information from the New Haven Harbor tidal benchmark and methods outlined in the Pooled Fund Study SPR-3(22) on tidal hydraulics (Ayres Associates, 1997). Data from the Corps of Engineers ADCIRC-2DDI storm surge prediction model is used to develop stage graphs. The ADCIRC station nearest to the crossing is station 368. However, because storm surges for the Connecticut coastline were not computed as part of the ADCIRC project, maximum storm tide elevations are taken directly from the FEMA reports. The 50-year, 100-year, and 500-year peak storm surge elevations for New Haven Harbor, referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29), are 3.05m (10ft), 3.23m (10.6ft), and 3.75m (12.3ft). (Referenced to NAVD 88, the peak storm surge elevations are 2.73m, 2.94m, and 3.46m for the 50-year, 100-year, and 500-year return periods, respectively.) The FEMA studies for both New Haven and West Haven estimate a 100-year stillwater elevation of 10.7 feet NGVD for the West River. FEMA's estimates are based on data provided in NOAA technical report NWS-38 (National Weather Service, 1987).

Analysis of the NOAA tide gage in New Haven Harbor shows that the mean tidal amplitude is 2.07m (6.78ft), with a mean lower low water elevation 0.83m (2.72ft) below the NGVD 29. Combining the storm surge with the normal tides produces the 100-year and 500-year stage-graphs shown in Figures 3 and 4, respectively.

Storm surges for fourteen historical storm events from the Corps of Engineers Surge Database for ADCIRC station 368 are included in the Pooled Fund Study report. The highest observed water level at the New Haven Harbor tidal gage was 1.63m (5.36ft) above NGVD 29 on September 27, 1988. The lowest observed water level was 1.10m (3.62ft) below NGVD 29 on September 26, 1988. The New Haven FIS also documents significant events occurring in 1815, 1938, 1944, 1955, and 1960.

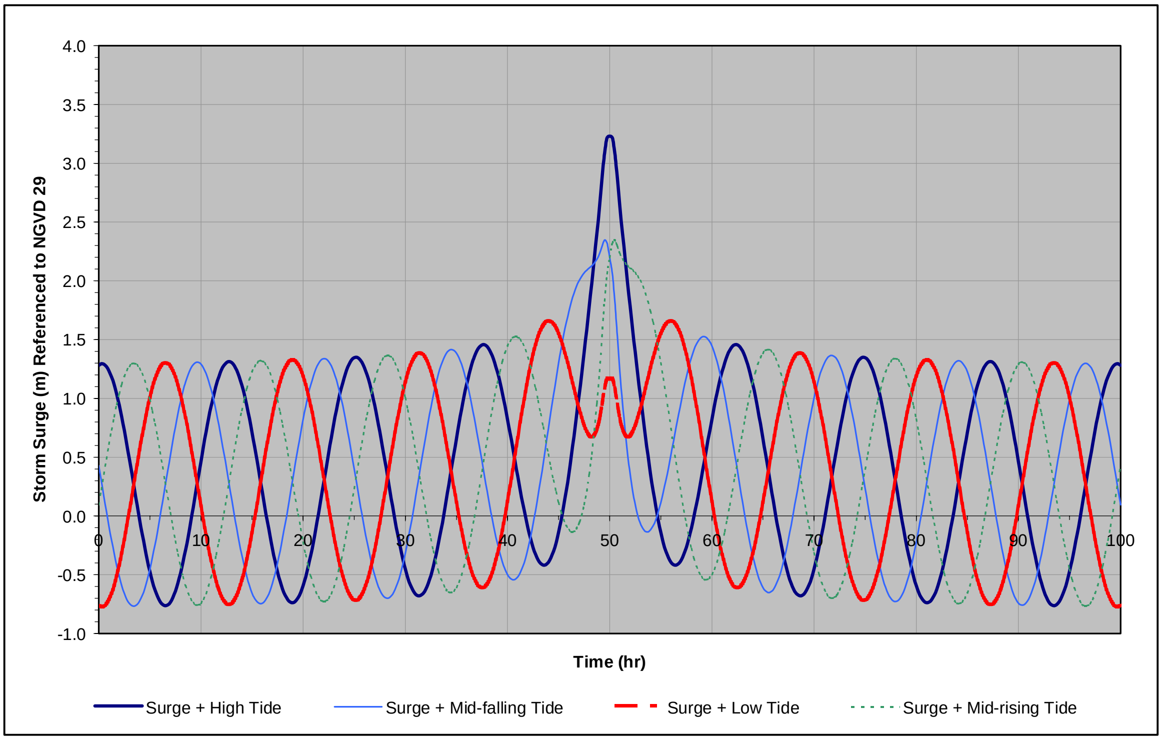

Figure 3: 100-Year Return Period Storm Surge

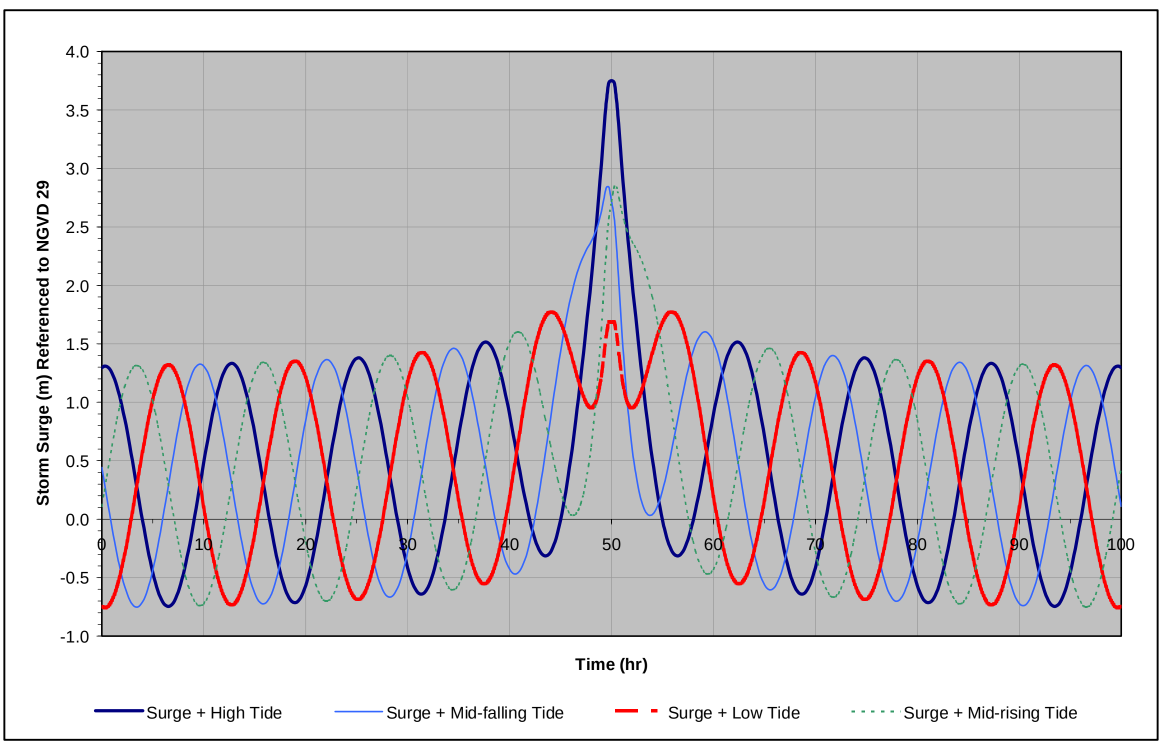

Figure 4: 500-Year Return Period Storm Surge

Both riverine and tidal controlling storm events were considered as hydrologic boundary condition inputs into the FESWMS hydraulic model. Table 2 summarizes the boundary conditions for each storm type and storm event analyzed.

Table 2. Summary of FESWMS Model Boundary Conditions.

Flood Scenario	Riverine Boundary Condition (Steady Flow)	Tidal Boundary Condition (Unsteady Flow)
100-year Riverine Flood	100-year Riverine Discharge (136 cms)	Normal Tidal Cycle from Mean Higher High Water (1.24 m) to Mean Lower Low Water (-0.83 m above NGVD 1929)
100-year Tidal Flood	10-year Riverine Discharge (78 cms)	100-year Tidal Storm Surge Stage Graph
500-year Riverine Flood	500-year Riverine Discharge (178 cms)	Normal Tidal Cycle from Mean Higher High Water (1.24 m) to Mean Lower Low Water (-0.83 m above NGVD 1929)
500-year Tidal Flood	10-year Riverine Discharge (78 cms)	500-year Tidal Storm Surge Stage Graph

Additionally, each storm tide phasing was considered in the hydraulic model, e.g. storm surge peak coinciding with mid-rising, high, mid-falling or low tides. Analysis of the hydraulic model simulations for each of the four storm tide phasings showed that the combination of the storm surge peak coinciding with high tide produced the largest velocities in the West River at the I-95 Bridge.

TEMPORARY CONDITIONS

Hydraulic and scour analyses for final design incorporate the evaluation of temporary conditions at the Existing Piers and the Replacement Piers for the worst case “during construction” scenario. The 25-Year FEMA Flood Insurance Study event and the 25-year tidal storm surge were used to evaluate “during construction” conditions at the I-95 crossing.

DESIGN CRITERIA FOR TEMPORARY CONDITIONS

The procedure for establishing the design flood criteria for the temporary cofferdam systems needed during the construction of the replacement I-95 Bridge over West River is outlined in Section 6.15 and Appendix 6.F (Hydrology for Temporary Facilities) of the 2000 ConnDOT Drainage Manual. The design frequency is determined based on impact factors, which, at this location, are associated primarily with the Average Daily Traffic.

An Urban ADT exceeding 3000 has a rating of 3, and the rating for Potential Loss of Life is equal to 15 times the ADT rating. Taking into account the other selection factors of Drainage Area, Height Above Streambed, Detour Length, Traffic Interruption, and Property Damages, the Total Impact Rating is greater than 50. The design flood for temporary structures is then determined from the charts in Figure 5 (taken from Appendix 6.F) to be the 25-year event, based on the length of construction.

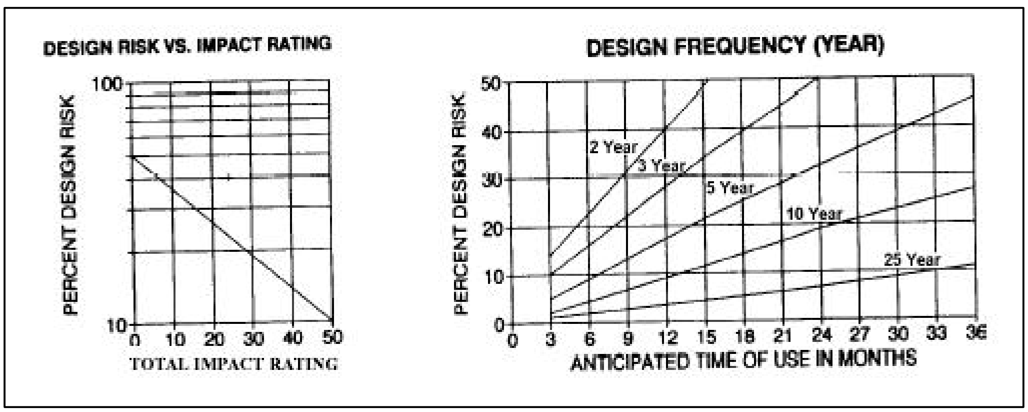

Figure 5: ConnDOT charts for determining design frequency for temporary structures

The FEMA flood elevations, referenced to NAVD 1988, are

10-year	2.27m
50-year	2.73m
100-year	2.94m
500-year	3.46m

The 25-year flood elevation is 2.54m NAVD 1988.

HIGH TIDE LINE

The High Tide Line (HTL) elevation was established in order to ensure that potential underwater obstructions from the substructure of the replacement bridge remain visible above the water line. The HTL for the replacement I-95 Bridge over West River was determined using data published by the National Oceanic and Atmospheric Administration (NOAA) for Tidal Bench Marks 8465748 (New Haven, New Haven Harbor) and 8467150 (Bridgeport Harbor). Four months of data were used to determine the tidal datums at the New Haven Harbor gage, which was in place from May 5, 1988 through October 15, 1988. The Bridgeport gage has been in place since June of 1932, and is used as a reference for predicting tides at the New Haven gage. Table 3 shows the tidal datums published for these gages, in meters referenced to NAVD 88. Table 4 shows the correction factors for converting predicted tides from the Bridgeport gage to the New Haven gage.

TABLE 3: TIDAL DATUMS
(Meters NAVD88)

Datum	New Haven	Bridgeport
Mean Higher High Water	0.92	1.01
Mean High Water	0.82	0.91
Mean Tide Level	-0.12	-0.12
Mean Low Water	-1.07	-1.15
Mean Lower Low Water	-1.15	-1.23

TABLE 4: TIDAL CORRECTION FACTORS

Station	Time Difference		Height Difference		Reference Station
	High	Low	High	Low	
New Haven Harbor Entrance	-0:09	-0:14	*0.92	*0.92	Bridgeport
New Haven (City Dock)	+0:01	-0:01	*0.89	*0.88	Bridgeport

To convert from MLLW at Bridgeport to NAVD88 at New Haven, the value is first multiplied by 0.89 and then 1.15m are subtracted.

Observed 6-minute tide levels dating back to January 1, 1996 are available online at http://co-ops.nos.noaa.gov/data_res.html for the Bridgeport gage; monthly extremes are also available, dating back to 1965. Tide predictions for the Bridgeport gage, dating back to January 1, 1800, can be obtained at http://co-ops.nos.noaa.gov/tp4days.html. Observed daily high tides for January 1996 through December 2001 were compared with predicted daily high tides over the same period. The data is summarized in Table 5.

TABLE 5: ANNUAL HIGH TIDES AT BRIDGEPORT (Meters above MLLW)					
Year	Annual Predicted Maximum	Annual Observed Maximum	# Days Above Predicted Max	Day Observed	Time Observed
1996	2.6	3.260	61	10/19/1996	21:48
1997	2.638	3.139	43	8/21/1997	18:06
1998	2.649	3.024	41	2/24/1998	14:12
1999	2.659	2.969	15	1/3/1999	16:48
2000	2.597	2.905	46	11/10/2000	14:42
2001	2.598	3.000	36	3/7/2001	14:00

The average predicted annual maximum high tide for the Bridgeport gage is 2.62m MLLW (1.18m NAVD88 at New Haven). However, observed values over the 6-year period exceeded the predicted annual maximum high tide on 230 days, or 10% of the time. The average observed annual maximum tide is 3.05m MLLW (1.56m NAVD88 at New Haven), which was exceeded only 4 days in the 6-year period.

Comparisons of the daily predicted and observed data suggest that the observed data should be used in determining the High Tide Line. Historical weather records for New Haven for the days of observed annual maximum high tides (http://www.wunderground.com/US/CT/New_Haven.html), from 1996 to 2001, indicate rain, with winds ranging from 10.2mph to 17.7mph. Taking the average of the annual maximum tides minimizes the impact of any outliers caused by storm surges or intense storms.

Based on the analysis of the 6 years of daily data, observed monthly extremes over the period 1983 to 2001 were used to determine the average annual observed maximum tide elevation at the Bridgeport gage. Over the 19-year period, the average annual maximum was 3.07m MLLW (1.56m NAVD88 at New Haven). 1.56m NAVD88 is recommended as the High Tide Line at the I-95 bridge over West River.

CONSTRUCTION STAGING

Construction of the I-95 Bridge Replacement will take place in several stages. Temporary trestles, sheet piling enclosures, and cofferdams will be used in Stages 1 through 3. The initial stage involves the use of cofferdams and trestles for the construction of the southern third of Replacement Piers 1 through 4. Sheet piling enclosures and temporary trestles for the demolition of the piers along the existing exit ramp to the east of the bridge are also in place during Stage 1.

During Stage 2, the trestles and cofferdams are extended for the construction of the middle third of Replacement Piers 1 through 4, and sheet piling enclosures are in place for the demolition of the southern half of Existing Piers 2 through 7. In Stage 3, the trestles and cofferdams are again extended for the completion of Replacement Piers 1 through 4. Sheet piling enclosures are also in place for the demolition of the northern half of Existing Piers 2 through 7. Construction Stages 1 through 3 are shown in Figures 6, 7, 8, and 9.

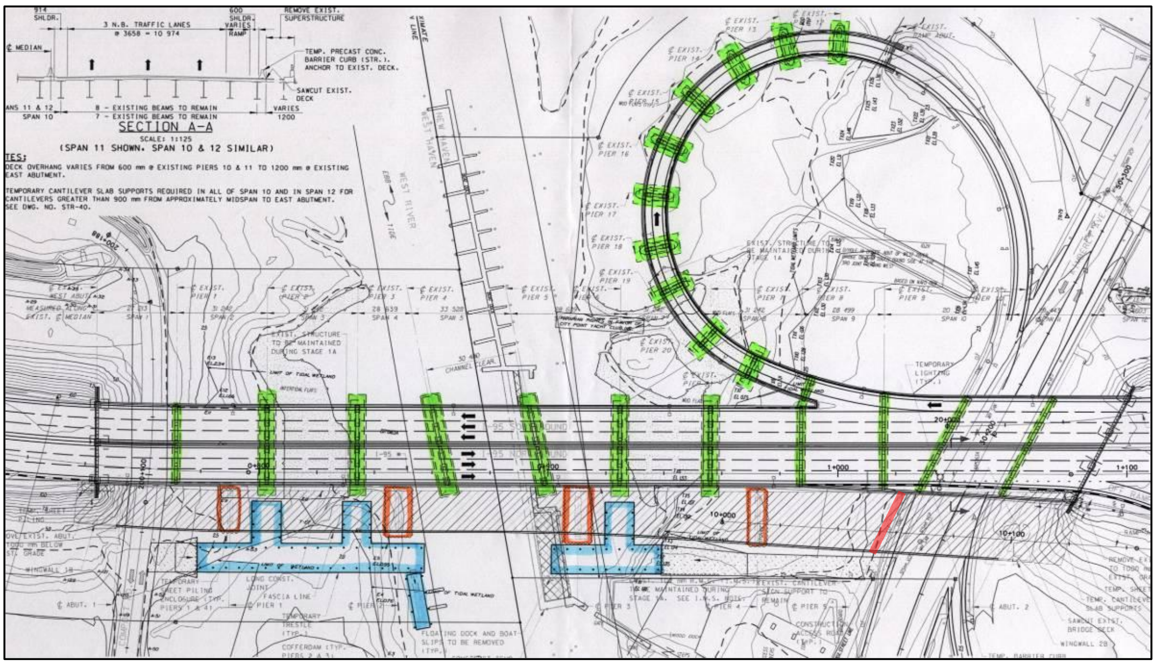

Figure 6: Construction Stage 1A. Existing piers are shown in green, cofferdams are shown in orange, trestles are shown in blue, and replacement Pier 5 is shown in red

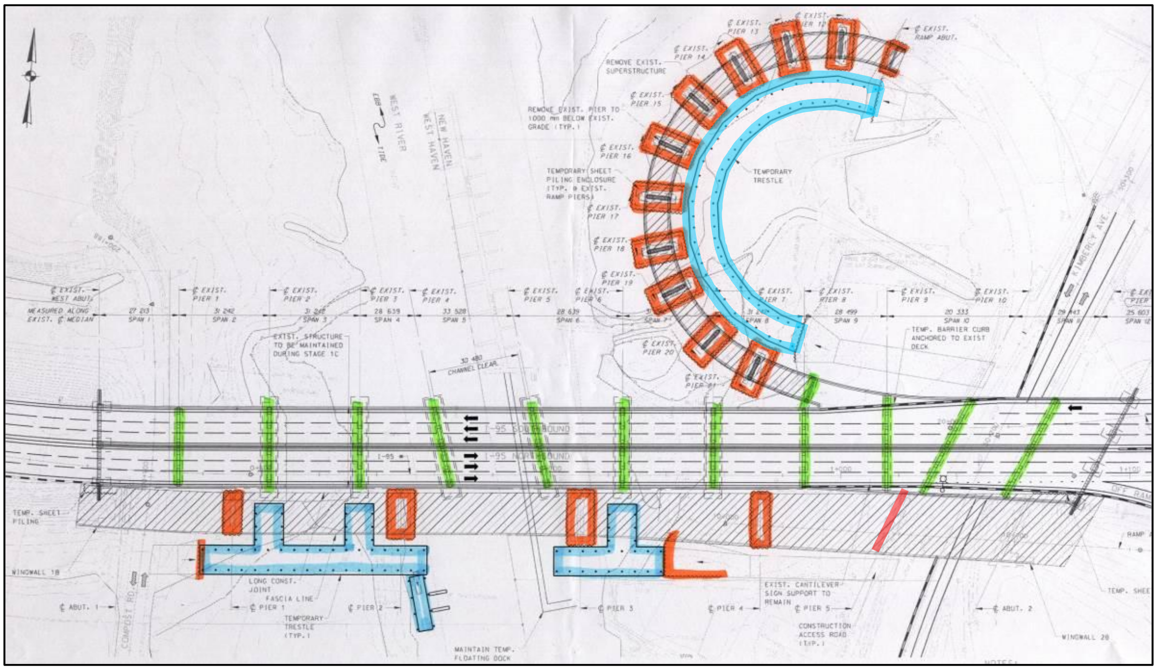

Figure 7: Construction Stage 1C

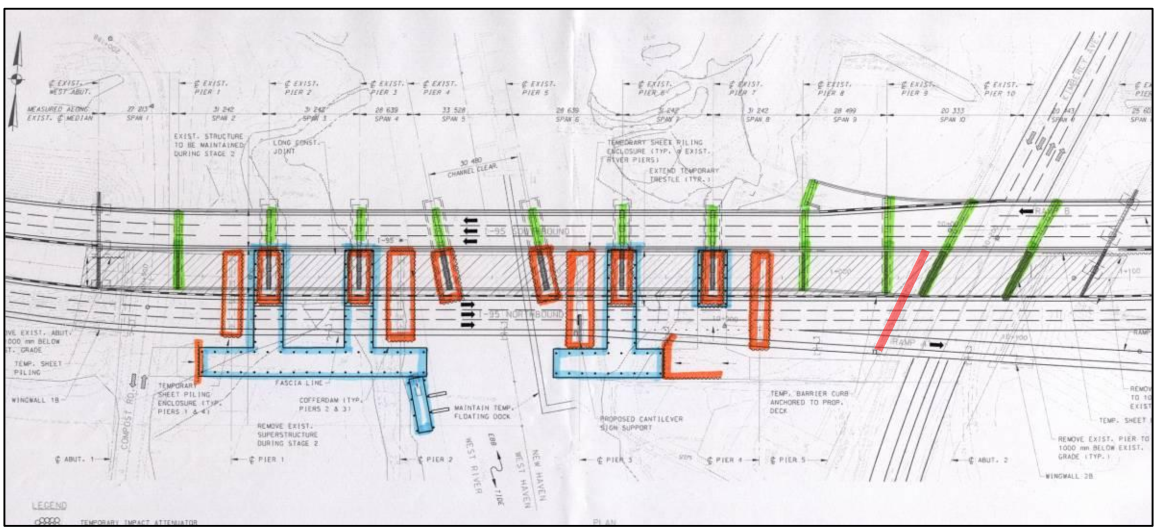

Figure 8: Construction Stage 2

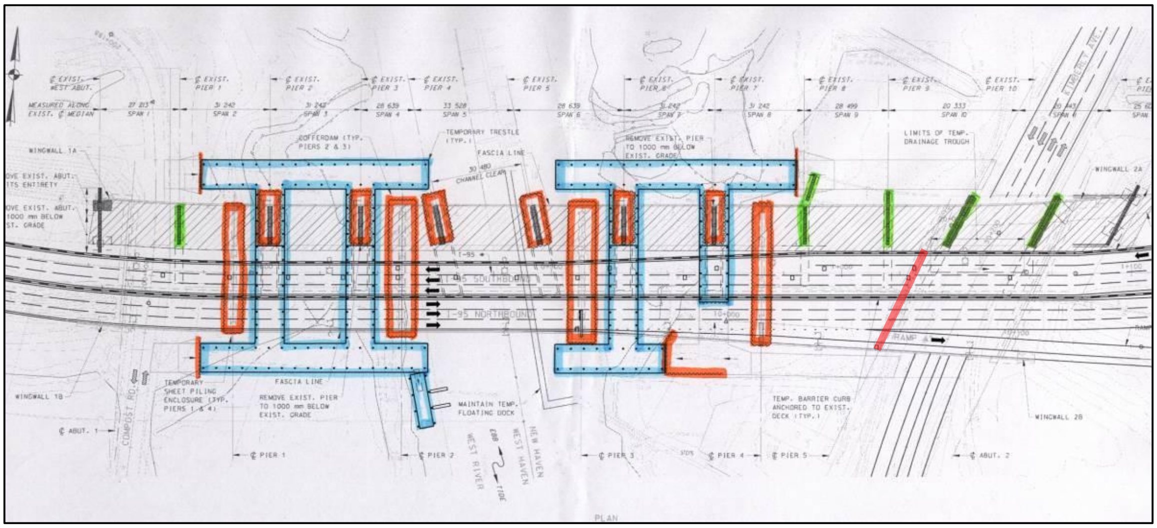

Figure 9: Construction Stage 3

HYDRAULIC ANALYSIS

MODEL SELECTION

Hydraulic modeling of the I-95 crossing requires a dynamic hydraulic model to simulate the tidal storm surge. Version 3.0 of the FHWA two-dimensional hydraulic model FESWMS-2DH (Froehlich, 2001) is used to model the tidal storm surge. FESWMS-2DH employs the finite element network method. The study area or "solution domain" is represented in FESWMS-2DH by the finite element network. The network is comprised of a series of interconnected elements. Elements may be triangular or quadrangular in shape. Elements are used to describe the study area, and are assigned hydraulic parameters such as Manning's roughness coefficient using property codes. Corner and mid-side nodes define the locations and elevations of elements. Each node has x-, y-, and z- coordinates. FESWMS-2DH solves the momentum and energy equations and computes the direction and depth of flow at each node point in the finite element network.

NETWORK LAYOUT

Solution Domain

The solution domain for the finite element network is depicted in Figures 10a and 10b. The network starts in New Haven Harbor a short distance outside the Kimberly Avenue Bridge and the mouth of the West River. It was not necessary to model more of the harbor as the flow between the West River and New Haven Harbor is restricted by the narrow opening of the Kimberly Avenue Bridge. The model extends through the West River upstream of the I-95 crossing to the tide gates at Orange Avenue/Congress Avenue.

Element Properties

Element property types are used to represent differing areas of hydraulic properties or to distinguish differing land features. The element property types used to model the I-95 crossing include:

- WEST_RIVER: This element type is used to represent the open water section of the West River extending from the Kimberly Avenue Bridge up to the tide gate. Field observations characterize the flow through the riverine section as hydraulically efficient.
- HARBOR_CHANNEL: This element type is used to represent the dredged, deep channel that connects the West River with the main navigation channel in New Haven Harbor.
- HARBOR_FLAT: This element type is used to represent the broad, shallow areas in New Haven Harbor adjacent to the mouth of the West River.

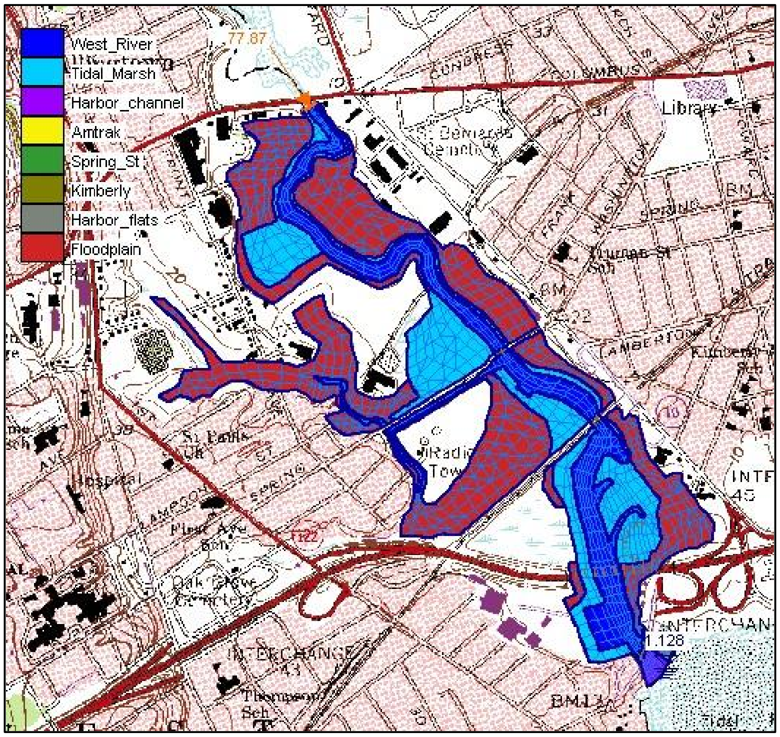

Figure 10(a): West River FESWMS-2DH Model Element Property Groups

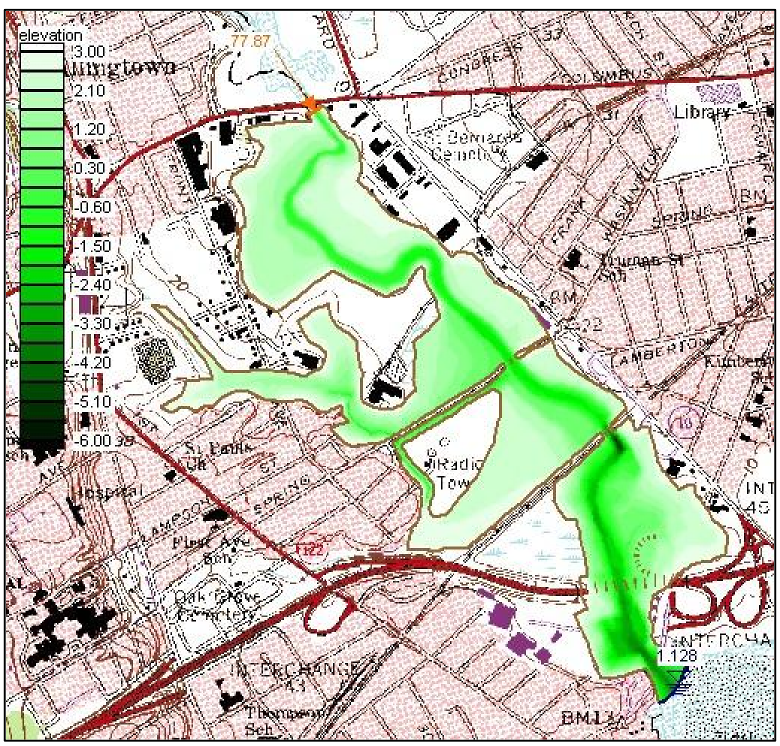

Figure 10(b): West River FESWMS-2DH Model Bathymetry

- TIDAL_MARSH: This element type represents the frequently-flooded tidal marshes on the margins of the West River upstream of the Kimberly Avenue Bridge. Generally, the maximum elevation of elements classified as tidal marsh is 2.8m. Mean high water is 1.14m and the 100-year flood elevation is 3.25m.
- AMTRAK, SPRING_ST, KIMBERLY, I-95: These four element types are used to represent the four bridges in the solution domain. AMTRAK and SPRING_ST are coded to allow pressure flow. No pressure flow occurs at the Kimberly Avenue Bridge or the I-95 Bridge.

Hydraulic material properties, such as eddy viscosity and Manning's n values, are input into the FESWMS model according to element material types. The eddy viscosity values chosen for the West River model was a constant 5 m^2/sec throughout the entire model. This eddy viscosity value was chosen based on the guidance in the FESWMS User Manual (Froehlich, 2001). FESWMS requires a low flow deph and a high flow depth Manning's n value. The depth intervals and n values are both user inputs. The Manning roughness coefficients chosen for the West River FESWMS model are documented in Table 6.

Table 6. Manning's Roughness Coefficients.

Material Type	Low Stage n	Low Stage Depth (m)	High Stage n	High Stage Depth (m)
WEST_RIVER	0.0285	1.0	0.0235	4.0
HARBOR_CHANNEL	0.016	1.0	0.016	4.0
HARBOR_FLAT	0.016	1.0	0.016	4.0
TIDAL_MARSH	0.100	1.0	0.065	2.0
AMTRAK, SPRING_ST, KIMBERLY, I-95	0.020	1.0	0.018	4.0

The Manning's roughness coefficients for the West River FESWMS model were selected following the guidance set forth in the "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" (Arcement, 1984). These n values were selected based on bed and bank material types, and vegetation, as the 2-D computations in FESWMS handle other channel properties commonly considered in n value selection.

Network Development

The network was developed using the NGVD 1929 vertical datum and the UTM Zone 18 coordinate system. Bridge plans were obtained for Kimberly Avenue, the existing I-95 Bridge, and the Amtrak Bridge. Plans for the tidal gates at Congress Avenue were supplied by the City of New Haven. Field measurements were made to obtain the dimensions of the Spring St. Bridge.

Detailed survey data was obtained for the West River from the Kimberly Avenue Bridge upstream to the Amtrak Rail Bridge. Bathymetric data for the area of New Haven Harbor was obtained from the NOAA navigation chart for new Haven Harbor. Limited topographic data was available for the area of overbanks and channels upstream of the Amtrak Bridge. Limits of channels and tidal marshes are clearly delineated on aerial photos, but elevations on USGS topography and other mapping possessed limited resolution. Bathymetric and ground topographic data for areas with poor resolution was developed by extrapolation from bridge plans and detailed surveys based on locations of vegetation, land use, and channel boundaries.

The network was developed using SMS Version 7.0. An initial network was developed and tested using the 500-year storm surge simulation. Successive runs identified areas requiring network adjustments or refinement. Adjustments were made to produce a smooth simulation.

Bridges

There are four bridge crossings through the solution domain that have significant impacts to flow.

<u>Kimberly Avenue</u>

The Kimberly Avenue Bridge has three piers in the water and contracts the flow through vertical abutments. The piers are small compared to the opening so they are represented by pier cards. Pier cards code the pier shape, size, and location. FESWMS-2DH uses the pier cards to compute the drag associated with the pier. There is no potential for pressure flow, so no treatment is required for the bridge deck.

<u>I-95</u>

The proposed I-95 crossing has five piers. The piers are small compared to the hydraulic opening, so they are represented by pier cards. There is no potential for pressure flow, so no treatment is required for the bridge deck. The abutment bases are at about 7m elevation and are set back from the edge of the floodplain. The 500-year storm surge elevation is less than 4m elevation, so the abutments will not come into contact with the storm surge. Abutments are not included in the model. The existing conditions finite element network is updated to represent the proposed bridge configuration by substituting alternate sets of pier cards that represent the proposed conditions.

Amtrak

The Amtrak crossing has a restrictive opening similar to a bottomless box culvert. The embankment is above elevation 4.0m, so there is no overtopping. Elements representing the bridge deck are coded with a ceiling elevation and the element property type representing the bridge is coded to allow pressure flow. When flow elevations exceed the ceiling elevation, FESWMS-2DH automatically switches to pressure flow computations if the element property type is coded to allow pressure flow.

Spring St.

The Spring St. crossing has a restrictive opening similar to a bottomless box culvert. The bridge is perched over the West River. There is a low spot on the West Haven approach where flow can travel across the roadway. The Spring St. Bridge was treated as a bridge rather than a culvert. Elements representing the bridge were coded with a ceiling elevation and the element property type representing the bridge is coded to allow pressure flow. When flow elevations exceed the ceiling elevation, FESWMS-2DH automatically switches to pressure flow computations if the element property type is coded to allow pressure flow.

Modeling of Construction Conditions

Hydraulic conditions at the crossing were evaluated for the scenario where the maximum amount of river obstruction will be created by the Existing and Replacement Piers, and the temporary trestles and cofferdams. Stage 2 and Stage 3 construction will present the greatest obstruction to flow in West River. Stage 2 and Stage 3 Construction are shown in Figures 11 and 12, respectively. The FESWMS model was modified to reflect the temporary conditions during Stage 2 and Stage 3 construction of the replacement bridge. The following modifications were made:

Stage 2 – Existing piers still in use for southbound traffic

- East and West trestles in place south of I-95 (not shown)
- Cofferdams in place around southern 2/3 of Proposed Piers 1 through 4
- Existing Piers 1,8,9,10,11, and 22 in place
- Sheet Piling Enclosures in place around southern half of Existing Piers 2-7
- Northern half of Existing Piers 2-7 in place
- Southern 2/3 of Proposed Pier 5 in place
- Existing Piers 12-21 removed

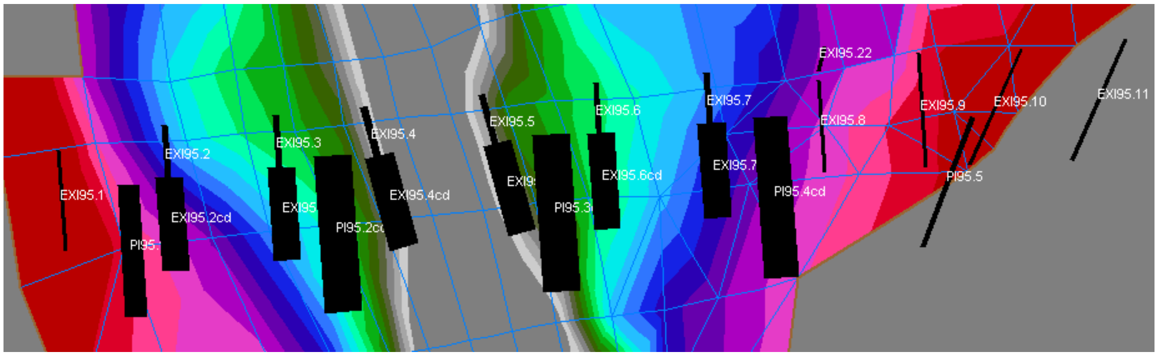

Figure 11: Model of Stage 2 Construction Conditions

<u>Stage 3 – All traffic on southern 2/3 of Proposed piers</u>

- East and West trestles in place on north and south sides of I-95 (not shown)
- Cofferdams in place around Proposed Piers 1 through 4
- Southern half of Existing Piers 1-11 removed
- Northern half of Existing Piers 1, 8, 9, 10, 11 in place
- Existing Pier 22 in place
- Proposed pier 5 in place
- Sheet piling enclosures in place around northern half of Existing Piers 2-7

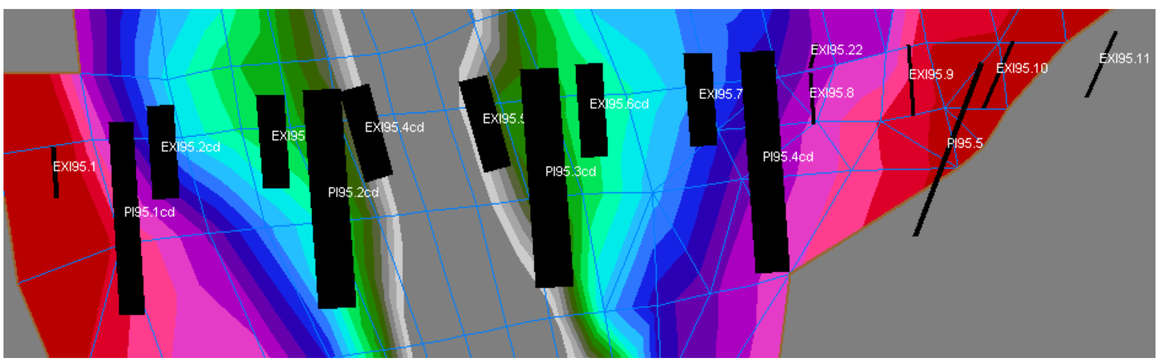

Figure 12: Model of Stage 3 Construction Conditions

MODEL CALIBRATION

There is no flow data available to calibrate the model. Boundary conditions are based on tidal stage recorded in New Haven Harbor. Model adjustments were made to produce reasonable simulation results. The specified maximum stages are produced at the I-95 Bridge.

MODEL SIMULATIONS

FESWMS-2DH was run to simulate the four scenarios: (1) the scour design event; (2) the scour check event; (3) the 25-year return period discharge with Stage 2I construction; and (4) the 25-year return period discharge with Stage 3 construction. The model output specifies the depth, direction, and flow velocity throughout the finite element network. The results are used to estimate scour at the bridge piers in the scour analysis.

Tidal Simulations

Tidal storm surge simulations were modeled using FESWMS-2DH for the 25-year, 100-year and 500-year storm surges. Following ConnDOT procedure, the upstream inflow from the West River was set to the 10-year return period discharge.

Riverine Simulations

At the direction of ConnDOT, simulations were performed for the 100-year and 500-year riverine discharges. FEMA 100-year and 500-year discharges were assigned to the upstream West River boundary. The simulations were run through two 12.6-hour tidal cycles. For construction conditions, the 25-year riverine discharge was applied to the upstream model boundary.

Simulation Results

After each simulation has been run, FESWMS creates a *.flo file containing all of the computed hydraulic information for each point in the model grid. The hydraulic parameters that are included in this file are: depth averaged velocities in the x and y direction, water-surface elevation, and time derivatives of each value. The data from the flo file is extracted at specific network locations for use in the local scour analysis using a proprietary PBQ&D program. In the extraction process the x and y velocity values are combined to produce a net velocity and flow angle at each location. Another PBQ&D program is used to compute flow rates through a specified cross-section to produce the necessary inputs for contraction scour analyses.

The resultant maximum water surface elevations at the I-95 crossing at presented in Table 7. These water surface elevations are as predicted by the FESWMS hydraulic model that has been developed and calibrated for scour analysis. The water surface elevations are expected to be lower than the water surface elevations predicted by the one-dimensional HEC-RAS model developed for flood plain certification purposes.

Table 7. Maximum Water Surface Elevations at the I-95 Crossing.			
Model Condition	Event	Maximum Water Surface Elevation (meters above NGVD 1929)	
		Tidal Flood	Riverine Flood
Existing Conditions	100-year	3.221	1.295
Existing Conditions	500-year	3.737	1.319
Proposed Conditions	100-year	3.221	1.295
Proposed Conditions	500-year	3.732	1.322
Stage II Construction	25-year	2.847	1.274
Stage III Construction	25-year	2.847	1.274

SCOUR ANALYSIS

INTRODUCTION

Scour evaluations are performed in accordance with FHWA guidelines for scour presented in the fourth edition of Hydraulic Engineering Circular 18: *Evaluating Scour at Bridges* (HEC-18). Under HEC-18 guidelines, total scour at a bridge is composed of three components: (1) long term aggradation or degradation; (2) contraction scour; and (3) local scour. Each of the scour components is assumed to occur separately. The total scour is computed by adding the separate scour components together.

LONG-TERM SCOUR

Long-term scour addresses how long-term trends in aggradation or degradation will impact the crossing. The West River appears to be stable and adjusted to urbanization. Examination of the borings taken when the I-95 crossing was originally constructed shows that deep sediments are composed of sand, while surficial sediments are primarily silts and mud. This is evidence that the channel is depositional. New sediment entering the river is limited to suspended sediment passing over the tidal gates at Congress Avenue or brought in by tidal action. It is concluded that the West River tidal zone is probably aggrading, but at a slow rate. Therefore, long-term scour trends are assumed to be negligible.

CONTRACTION SCOUR

Contraction scour occurs when the flow area of a stream at flood stage is reduced by the presence of bridge piers and abutments within the channel transporting flows. The reduction in flow area causes a local increase in flow velocities and sediment transport capacity. Contraction scour analyses are based upon the principle of conservation of sediment. The analyses assume that scour will occur at the bridge until the cross sectional area increases sufficiently to reduce the flow velocity below the level at which sediment may be transported.

The potential for contraction scour at the I-95 crossing is minor. The abutments of the existing I-95 bridge are set back from West River and do not cause a flow contraction at flood elevations. There is no pressure scour because the bottom of the bridge deck is well above the 500-year flood elevation. Similarly, the abutments for the proposed bridge are all out of the 500-year flood plain except for the east abutment (New Haven). There is a slight flow contraction created by the piers that block a small portion of flow area.

The first step in the analysis is to assess the flow's ability to transport sediment in the reach above the bridge. The critical velocity, the velocity required to mobilize sediment from the bottom, is computed by HEC-18 Equation (13):

$$v_c = 6.19\, y^{1/6} D^{1/3} \qquad (1)$$

where: v_c = the critical velocity above which bed material of size D and smaller will be transported

y = the flow depth

D = the grain diameter of bed sediment.

Determination of critical velocity requires estimates of bottom sediment diameter, flow velocity, and flow depth. If the flow velocity is above the critical velocity then live-bed scour occurs. If the flow is below critical velocity then clear-water scour occurs.

The median particle diameter (D_{50}) is usually used to represent the average properties of the bottom sediment. Several soil borings were taken in the vicinity of the I-95 crossing over the West River. Analysis of these soil borings shows that the channel bed material throughout the anticipated depth of scour consists primarily of organic silts with traces of fine sand. Gradation analysis of this material produces D_{50} values in the silt range with a representative value of 0.016 mm chosen. Additionally, the gradation analysis showed D_{84} values also in the silt range with a representative value of 0.08 mm and D_{95} values in the fine sand range with a representative value of 0.13 mm chosen. These results confirm the observation that the channel material primarily consists of silt with traces of fine sand. Calculations show that silt is easily scourable by even a small velocity. Thus the I-95 crossing of the West River is subject to live bed scour.

The live-bed contraction equation assumes that contraction scour occurs until the hydraulic conditions at the bridge reach equilibrium with the hydraulic conditions upstream from the bridge. For example, a reduction in flow area at the bridge from the upstream channel causes increased flow velocities at the bridge; scour occurs and increases the bridge cross section until flow velocities and sediment transports at the bridge match the upstream channel. The live-bed contraction scour equation is:

$$\frac{y_2}{y_1} = \left[\frac{Q_2}{Q_1}\right]^{6/7} \left[\frac{W_1}{W_2}\right]^{k_1} \qquad (2)$$

where: y_1 = average depth in upstream main channel;

y_2 = average depth in contracted section after scour occurs;

y_0 = existing depth in contracted section before scour occurs;

Q_1 = flow in upstream main channel transporting sediment;

Q_2 = flow in contracted section;

W_1 = top width of upstream main channel transporting sediment;

W_2 = top width of the contracted section less pier widths;

k_1 = 0.69 (suspended sediment transport - coefficient based on mode of bed material transport (See HEC-18); and

y_c = scour depth = $y_c - y_0$.

The upstream section is taken as the section directly upstream of the bridge. The discharge upstream and at the I-95 crossing are assumed to be equal.

Calculations are shown in Appendix A. Computations are made for riverine and tidal storm surge events. Tidal contraction scour is greater than the riverine contraction scour because the extent of riverine flooding is limited to areas adjacent to the main channel. Results of contraction scour are shown in Table 8. The bridge replacement will have little impact on contraction scour, which will decrease slightly for the 100-year tidal event, but will increase for the 500-year tidal event because Pier 5 is not aligned with the direction of flow.

TABLE 8: CONTRACTION SCOUR RESULTS

Model Conditions	Tidal Storm Surge Contraction Scour Depth (m)			Riverine Flood Contraction Scour Depth (m)		
	25-year	100-year	500-year	25-year	100-year	500-year
Existing Bridge	--	0.48	0.54	--	0.19	0.19
Replacement Bridge	--	0.28	0.60	--	0.07	0.07
Stage 2 Construction	0.8	--	--	1.0	--	--
Stage 3 Construction	2.0	--	--	0.8	--	--

LOCAL SCOUR

Abutment Scour

The abutments of the existing I-95 bridge are set back from West River and are above the elevation of the 500-year storm surge. The west abutment for the proposed bridge is above the elevation of the 500-year flood, but the east abutment (New Haven) of the proposed bridge is in the floodplain. However, the base of the concrete slope protection for the east abutment is below the 100-year and 500-year storm surge elevations. Flow velocities at peak storm surge elevations are close to zero, especially at the margins of the floodplain where the abutment is located. Significant flow velocities do not occur until tidal stages drop below 2m. Because the depth of flow is shallow and the slope will have erosion protection, abutment scour is assumed to be zero.

Pier Scour

The basic pier scour equation in HEC-18 is called the CSU equation. The CSU equation accounts for the orientation of the pier with respect to flow, flow velocities and depth, pier dimension and shape, and bed conditions. The CSU equation was developed using simple pier shapes such as columns or uniform rectangular shapes. As experience with HEC-18 and the scour evaluation program developed, additional research has been performed to adapt the basic CSU equation to treat complex pier shapes. The fourth edition of HEC-18 presents the most recent adaptation of the CSU equation for evaluating scour at complex piers. In our first round of analyses, the Complex Pier method was used to evaluate the scour generated by piers composed of a pier bent or pier wall placed atop a pier plinth. Analysis of the riverine floods showed that the pier plinths were not overtopped. Analysis of tidal storm surges showed that the plinths were overtopped, but that peak scour potential did not occur until tidal stages dropped below the top of the pier plinths. Thus, the standard CSU equation was applicable for all local pier scour estimates.

<u>Existing I-95 Bridge</u>

Piers at the existing I-95 Bridge are classified as both complex and simple. There are nine piers exposed to the 100-year and 500-year floods. Piers 2-7 consist of a pile supported footer, plinth, and pier bent. Piers 8 –10 consist of a pier bent atop a pile supported footer. Footings for all piers are buried and below the depth of contraction scour. Initially, scour for Piers 2-7 was calculated using the Complex Pier method with the plinths treated as the footing components. Review of scour results showed that at the time of maximum scour, water levels had dropped below the top of the plinths so that only the plinths were exposed to flow. As a result, scour was recomputed using the standard CSU equation. Scour for Piers 8-11 is evaluated using the standard CSU equation. Only the pier columns are subject to scour, so the empty spaces between columns are ignored.

<u>Replacement I-95 Bridge</u>

The proposed I-95 Bridge consists of five replacement piers of similar configuration to those of the existing bridge. The piers will possess buried footings. In the preliminary scour analysis, scour at the five piers was determined using the standard CSU equation. For final design, however, the complex pier method was implemented to fully account for any impacts to scour in the event that the footings become exposed. Piers 1, 4, and 5 consist of pier bents on pile supported footers. The majority of the time, only the pier bents will are exposed to flow. The pier bents of Piers 2 and 3 sit on plinths atop pile supported footers. For these piers, the plinths are exposed to most of the flow. During the 100-year and 500-year tidal storm surge events, scour depths go below the top of the footers at Piers 2 and 3.

<u>Temporary Construction Conditions</u>

During construction, sheet piling enclosures and cofferdams will be placed in stages around the existing and replacement piers. When construction is complete, the sheet piling around the replacement piers will be cut to the elevations of the tops of the footings. Scour was evaluated at the Existing Piers and the Replacement Piers for two “during construction” scenarios. The 25-year combined riverine and tidal discharge events were used to evaluate the scour during construction.

The cofferdams placed during construction will extend to the elevation of the 25-year tidal event, and will obstruct the flow completely. The sheet piling enclosure placed for demolition will extend to 300 mm above the high tide elevation of 1.56m. During Stage 2 construction, traffic will be maintained on the northern portion of the existing piers while sheet piling enclosures are in place around the southern portion of the existing piers. This creates a greater scour potential at the existing piers than under pre-construction conditions.

Pier Scour Results

Pier scour was computed for the 100-year and 500-year tidal and riverine events, and for the 25-year riverine and tidal events for temporary conditions. Scour computations are included in Appendix B. Computations were prepared using a spreadsheet. The spreadsheet was validated using the example computations provided in HEC-18. Hydraulic variables are taken from the FESMWS-2DH output. A macro was run to evaluate the scour for each time step of the model simulations. Hydraulic variables shown in the computations reflect the set of hydraulic variables from the time step that produced the maximum scour at each pier.

Review of scour analysis results shows that the maximum scour under tidal conditions occurs about two to three hours after the peak storm surge. Reviewing the hydraulic results shows that as the peak stage is reached during the storm surge, water velocities slow and become still before flow reverses and start flowing out. The stage in New Haven Harbor falls very quickly after the peak surge. Water levels remain high in the West River after the peak stage in New Haven Harbor because the Kimberly Avenue Bridge restricts the amount of flow leaving the West River. Flow starts to accelerate as the difference in stages between New Haven Harbor and the West River increases. Maximum flow velocities are produced about two hours after flow reversal. The maximum depth of pier scour occurs at about the same time that maximum flow velocities occur. Scour potential is high during a tidal storm surge because the flow is emptying off the floodplains and the direction of flow is not aligned with the bridge piers.

At some piers, scour potential is highest under riverine flooding conditions. This is the case for Existing Piers 4 and 5 for Stage 2 construction, and for Replacement Piers 2 and 3 for Stage 2 and Stage 3 construction.

TOTAL SCOUR

Total scour is the sum of long-term scour, contraction scour, pressure scour, and local scour. The total scour is subtracted from the ground elevation to determine the elevation of maximum scour. Ground elevations are the average ground elevation from the survey conducted in the spring of 2001. The depth of scour does not reach to bedrock at any of the piers or abutments. Summary pier scour results are reported in Tables 9 and 10.

The results listed in Tables 9 and 10 indicate that scour will reach elevations below the base elevation of footing and expose limited amounts of the piles at Replacement Pier 2 for the 100-year and 500-year tidal storm surge events. The piles at Replacement Pier 3 are also at risk for exposure during Construction Stage 3. This estimate does not take into account the effects of the cofferdams that will remain in place after construction. The piles of Replacement Piers 1 to 4 will be surrounded by sheet piling, and will not technically be exposed when scour depths go below the elevation of the base of the footings. The actual scour may therefore be less than estimated by HEC-18 procedures, but the reduction is not quantifiable with the available evaluation methods.

Scour potential at the existing piers during construction is significant, with up to 9.3 meters of scour estimated at Existing Pier 5 during Stage 2. This estimate does not take into account the effect of the cofferdam originally used in construction. The existing piers need to be monitored carefully during construction, particularly during and after any storm events. The implementation of a scour monitoring plan should be considered.

TABLE 9: TOTAL SCOUR RESULTS FOR EXISTING CONDITIONS										
Pier	Ground Elevation	Contraction Scour		Pier Scour		Total Scour		Elevation of Scour Worst Case	Bottom of Footer	Piles Exposed
		Tidal	Riverine	Tidal	Riverine	Tidal**	Riverine			
	m	m	m	m	m	m	m	m	m	(Y/N)
100-Year Return Period										
2	1.34	0.00	0.00	2.07	0.00	**2.07**	0.00	-0.73	-2.92	N
3	0.27	0.48	0.00	3.09	2.18	**3.57**	2.18	-3.30	-2.92	**Y**
4	-2.10	0.48	0.19	1.83	2.09	**2.31**	2.28	-4.41	-5.2	N
5	-1.45	0.48	0.19	3.43	4.06	3.91	**4.25**	-5.70	-5.2	**Y**
6	0.07	0.48	0.00	1.87	1.55	**2.36**	1.55	-2.29	-2.92	N
7	1.18	0.48	0.00	2.61	1.30	**3.09**	1.30	-1.91	-2.92	N
8	1.99	0.48	0.00	1.74	0.00	**2.22**	0.00	-0.23	0.13	**Y**
9	2.84	0.48	0.00	1.45	0.00	**1.93**	0.00	0.91	0.97	N
10	3.06	0.00	0.00	0.71	0.00	**0.71**	0.00	2.35	1.05	N
500-Year Return Period										
2	1.34	0.00	0.00	2.32	0.00	**2.32**	0.00	-0.98	-2.92	N
3	0.27	0.54	0.19	3.33	2.32	**3.87**	2.51	-3.60	-2.92	**Y**
4	-2.10	0.54	0.19	1.89	2.26	2.43	**2.45**	-4.55	-5.2	N
5	-1.45	0.54	0.19	3.42	4.41	3.96	**4.60**	-6.05	-5.2	**Y**
6	0.07	0.54	0.00	2.22	1.59	**2.76**	1.59	-2.69	-2.92	N
7	1.18	0.54	0.00	3.00	1.42	**3.54**	1.42	-2.36	-2.92	N
8	1.99	0.54	0.00	2.07	0.00	**2.61**	0.00	-0.62	0.13	**Y**
9	2.84	0.54	0.00	1.91	0.00	**2.45**	0.00	0.39	0.97	N
10	3.06	0.54	0.00	1.19	0.00	**1.73**	0.00	1.33	1.05	Y

TABLE 9: TOTAL SCOUR RESULTS FOR EXISTING CONDITIONS, CONTINUED										
Pier	Ground Elevation	Contraction Scour		Pier Scour		Total Scour		Elevation of Scour Worst Case	Bottom of Footer	Piles Exposed
		Tidal	Riverine	Tidal	Riverine	Tidal**	Riverine			
	m	m	m	m	m	m	m	m	m	(Y/N)
25-Year Return Period – Stage 2 Construction										
2	1.34	0.00	0.00	3.79	0.00	**3.79**	0.00	-2.45	-2.92	N
3	0.27	0.79	0.00	5.84	4.93	**6.63**	4.93	-6.36	-2.92	**Y**
4	-2.10	0.79	1.00	6.87	7.00	7.66	**8.00**	-10.10	-5.2	**Y**
5	-1.45	0.79	1.00	7.87	8.31	8.66	**9.31**	-10.76	-5.2	**Y**
6	0.07	0.79	1.00	4.62	3.78	**5.41**	4.77	-5.34	-2.92	**Y**
7	1.18	0.79	1.00	4.40	1.95	**5.19**	2.94	-4.01	-2.92	**Y**
8	1.99	0.00	1.00	1.46	0.00	**1.46**	1.00	0.53	0.13	N
9	2.84	0.00	1.00	0.49	0.00	0.49	**1.00**	1.84	0.97	N
10	3.06	0.00	1.00	0.00	0.00	0.00	**1.00**	2.06	1.05	N
25-Year Return Period – Stage 3 Construction										
2	1.34	0.00	0.00	3.47	0.00	**3.47**	0.00	-2.13	-2.92	N
3	0.27	1.99	0.00	5.08	3.79	**7.07**	3.79	-6.80	-2.92	**Y**
4	-2.10	1.99	0.77	6.79	6.75	**8.78**	7.52	-10.88	-5.2	**Y**
5	-1.45	1.99	0.77	7.46	7.62	**9.45**	8.39	-10.90	-5.2	**Y**
6	0.07	1.99	0.77	4.31	2.87	**6.30**	3.64	-6.23	-2.92	**Y**
7	1.18	1.99	0.77	4.07	1.80	**6.05**	2.57	-4.87	-2.92	**Y**
8	1.99	0.00	0.77	1.06	0.00	**1.06**	0.77	0.93	0.13	N
9	2.84	0.00	0.77	0.34	0.00	0.34	**0.77**	2.07	0.97	N
10	3.06	0.00	0.77	0.00	0.00	0.00	**0.77**	2.29	1.05	N

TABLE 10: TOTAL SCOUR RESULTS FOR PROPOSED CONDITIONS										
Pier	Ground Elevation	Contraction Scour		Pier Scour		Total Scour		Elevation of Scour Worst Case	Bottom of Footer	Piles Exposed
		Tidal	Riverine	Tidal	Riverine	Tidal**	Riverine			
	m	m	m	m	m	m	m	m	m	(Y/N)
100-Year Return Period										
1	2.69	0.28	0.00	0.57	0.00	**0.85**	0.00	1.84	-1.1	N
2	-0.12	0.28	0.07	5.86	3.46	**6.14**	3.53	-6.26	-4.1	**Y**
3	-0.55	0.28	0.07	2.33	1.29	**2.61**	1.36	-3.16	-4.1	N
4	1.79	0.28	0.00	1.44	0.00	**1.72**	0.00	0.07	-1.1	N
5	2.77	0.00	0.00	0.20	0.00	**0.20**	0.00	2.57	0.4	N
500-Year Return Period										
1	2.69	0.60	0.00	0.81	0.00	**1.42**	0.00	1.27	-1.1	N
2	-0.12	0.60	0.07	6.52	4.19	**7.12**	4.27	-7.24	-4.1	**Y**
3	-0.55	0.60	0.07	2.44	1.42	**3.05**	1.49	-3.60	-4.1	N
4	1.79	0.60	0.00	1.82	0.00	**2.42**	0.00	-0.63	-1.1	N
5	2.77	0.60	0.00	0.31	0.00	**0.91**	0.00	1.86	0.4	N
25-Year Return Period – Stage 2 Construction										
1	2.69	0.00	0.00	0.70	0.00	**0.70**	0.00	1.99	-1.1	N
2	-0.12	0.79	1.00	2.12	3.05	2.91	**4.04**	-4.16	-4.1	N
3	-0.55	0.79	1.00	1.90	2.53	2.69	**3.53**	-4.08	-4.1	N
4	1.79	0.00	0.00	1.42	0.00	**1.42**	0.00	0.37	-1.1	N
5	2.77	0.00	0.00	0.09	0.00	**0.09**	0.00	1.77	0.4	N
25-Year Return Period – Stage 3 Construction										
1	2.69	0.00	0.00	0.80	0.00	**0.80**	0.00	1.89	-1.1	N
2	-0.12	1.99	0.77	2.61	6.51	4.60	**7.28**	-7.40	-4.1	**Y**
3	-0.55	1.99	0.77	2.56	3.85	4.55	**4.62**	-5.17	-4.1	**Y**
4	1.79	0.00	0.00	1.45	0.00	**1.45**	0.00	0.34	-1.1	N
5	2.77	0.00	0.00	0.06	0.00	**0.06**	0.00	2.00	0.4	N

**Note: Peak tidal scour conditions all occur during the ebb surge or ebb tide with currents flowing downstream.

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APPENDIX A:

SCOUR ANALYSIS RESULTS

APPENDIX B:

CONTRACTION SCOUR COMPUTATIONS

100-year Tidal Storm Surge, Existing Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	299
W_2 = CONTRACTED SECTION WIDTH	m	3	227
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	2.29
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V^* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V^* / w			42.00
k_1			0.69
$(W_1 / W_2)^{K1}$			1.21
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.21
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**0.48**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

100-year Tidal Storm Surge, Proposed Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	299
W_2 = CONTRACTED SECTION WIDTH	m	3	254
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	2.29
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V* / w			42.00
k_1			0.69
$(W_1 / W_2)^{K1}$			1.12
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.12
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**0.28**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

500-year Tidal Storm Surge, Existing Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	315
W_2 = CONTRACTED SECTION WIDTH	m	3	243
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	2.75
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V^* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.06
V^* / w			46.08
k_1			0.69
$(W_1 / W_2)^{K1}$			1.20
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.20
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**0.54**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

500-year Tidal Storm Surge, Proposed Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	315
W_2 = CONTRACTED SECTION WIDTH	m	3	237
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	2.75
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V^* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.06
V^* / w			46.08
k_1			0.69
$(W_1 / W_2)^{K1}$			1.22
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.22
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**0.60**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

25-year Tidal Storm Surge, Stage II Construction

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	218
W_2 = CONTRACTED SECTION WIDTH	m	3	110
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	1.30
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.04
V* / w			31.68
k_1			0.69
$(W_1 / W_2)^{K1}$			1.61
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.61
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**0.79**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

25-year Tidal Storm Surge, Stage III Construction

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	1
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	1	1
W_1 = MAIN CHANNEL WIDTH	m	2	191
W_2 = CONTRACTED SECTION WIDTH	m	3	80
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	4	2.43
S_1 = ENERGY GRADELINE SLOPE	m/m	5	0.0001
D_{50} - BED MATERIAL	mm	6	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	7	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.06
V* / w			43.31
k_1			0.69
$(W_1 / W_2)^{K1}$			1.82
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.82
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	8	**1.99**

NOTES:

1. TIDAL FLOW UPSTREAM EQUIVALENT TO FLOW IN CONTRACTED SECTION
2. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
3. WIDTH AT CONTRACTED CHANNEL
4. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
5. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
6. OBTAIN FROM BORING OR GIVEN DATA.
7. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
8. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

100-year Riverine Flood, Existing Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	136
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	136
W_1 = MAIN CHANNEL WIDTH	m	3	154
W_2 = CONTRACTED SECTION WIDTH	m	4	132
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	1.70
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V^* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V^* / w			36.23
k_1			0.69
$(W_1 / W_2)^{K1}$			1.11
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.11
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**0.19**

NOTES:

1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

100-year Riverine Flood, Proposed Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	136
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	136
W_1 = MAIN CHANNEL WIDTH	m	3	154
W_2 = CONTRACTED SECTION WIDTH	m	4	145
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	1.70
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V* / w			36.23
k_1			0.69
$(W_1 / W_2)^{K1}$			1.04
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.04
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**0.07**

NOTES:

1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

500-year Riverine Flood, Existing Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	178
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	178
W_1 = MAIN CHANNEL WIDTH	m	3	154
W_2 = CONTRACTED SECTION WIDTH	m	4	132
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	1.70
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V* / w			36.23
k_1			0.69
$(W_1 / W_2)^{K1}$			1.11
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.11
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**0.19**

NOTES:
1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

500-year Riverine Flood, Proposed Conditions

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	178
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	178
W_1 = MAIN CHANNEL WIDTH	m	3	154
W_2 = CONTRACTED SECTION WIDTH	m	4	145
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	1.70
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V* / w			36.23
k_1			0.69
$(W_1 / W_2)^{K1}$			1.04
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.04
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**0.07**

NOTES:

1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

25-year Riverine Flood, Stage II Construction

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	97
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	97
W_1 = MAIN CHANNEL WIDTH	m	3	69
W_2 = CONTRACTED SECTION WIDTH	m	4	45
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	2.90
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.06
V* / w			47.32
k_1			0.69
$(W_1 / W_2)^{K1}$			1.34
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.34
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**1.00**

NOTES:
1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO $y_{1.}$

25-year Riverine Flood, Stage III Construction

CONTRACTION SCOUR COMPUTATIONS

LIVE-BED

	UNITS	NOTE	100 YEAR
Q_1 = FLOW IN MAIN CHANNEL	m^3/s	1	97
Q_2 = FLOW IN CONTRACTED SECTION	m^3/s	2	97
W_1 = MAIN CHANNEL WIDTH	m	3	69
W_2 = CONTRACTED SECTION WIDTH	m	4	45
y_1 = AVERAGE MAIN CHANNEL DEPTH	m	5	2.24
S_1 = ENERGY GRADELINE SLOPE	m/m	6	0.0001
D_{50} - BED MATERIAL	mm	7	0.016
w - FALL VELOCITY D_{50} BED MATERIAL	m/s	8	0.001
$(Q_2 / Q_1)^{6/7}$			1.00
V* = SHEAR VELOCITY = $[9.81(y_1)(S_1)]^{0.5}$	m/s		0.05
V* / w			41.59
k_1			0.69
$(W_1 / W_2)^{K1}$			1.34
$y_2/y_1 = (Q_2 / Q_1)^{(6/7)}(W_1 / W_2)^{K1}$			1.34
Y_s = SCOUR DEPTH = $y_2 - y_1$	m	9	**0.77**

NOTES:

1. FLOW IN UPSTREAM CHANNEL TRANSPORTING SEDIMENT
2. FLOW IN CONTRACTED CHANNEL (AT BRIDGE)
3. WIDTH OF CHANNEL TRANSPORTING SEDIMENT
4. WIDTH AT CONTRACTED CHANNEL
5. DEPTH OF FLOW IN CHANNEL TRANSPORTING SEDIMENT
6. SLOPE BETWEEN CHANNEL CARRYING SEDIMENT AND CONTRACTED SECTION
7. OBTAIN FROM BORING OR GIVEN DATA.
8. USING THE D_{50} VALUE AND FIGURE 3 IN THE HEC-18 MANUAL.
9. ASSUMES UNCONTRACTED DEPTH AT BRIDGE IS EQUAL TO y_1.

APPENDIX C:

PIER SCOUR COMPUTATIONS

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 100-YEAR RIVERINE FLOOD

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 21, 2003
Checked by	K. Brennan
Date	26-Jun-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Notes
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.000**	**26.875**	**31.000**	**43.000**	**47.375**	**25.750**	**51.000**	**51.000**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	1.20	1.01	-0.48	-0.42	0.49	1.27	1.20	1.20	1.21	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.08	0.41	1.60	1.75	0.21	0.10	0.08	0.07	0.06	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	1.87	13.53	2.10	12.22	14.04	28.76	44.27	46.84	19.92	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		0.90	1.00	0.90	1.00	1.00	1.00	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	
L	Length of pier	m	28.50	28.50	29.49	29.49	28.50	28.50	6.40	7.47	8.53	Ignores empty space between pier columns
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	-0.14	0.74	1.62	1.03	0.42	0.09	-0.79	-1.64	-1.85	z_w - zb
K_2	Correction factor for angle of attack		1.24	2.37	1.27	2.26	2.41	3.43	2.81	3.13	2.33	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		--	0.15	0.40	0.55	0.11	0.11	--	--	--	$V_1/(g^*y_1)^{.5}$
ys	Pier scour	m	--	2.18	2.09	4.06	1.55	1.30	--	--	--	$y_1{}^*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 500-YEAR RIVERINE FLOOD

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 21, 2003
Checked by	K. Brennan
Date	28-May-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Notes
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.000**	**26.875**	**43.875**	**30.500**	**35.625**	**25.875**	**51.000**	**51.000**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	1.23	1.05	-0.37	-0.29	0.82	1.27	1.23	1.24	1.24	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.09	0.49	1.84	1.95	0.26	0.13	0.09	0.08	0.07	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	0.77	13.07	2.22	12.74	10.38	28.01	42.20	45.64	19.35	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	$K_4 = 1$: Bed sediment $D_{90} \ll 20$ mm
Pier Data												
K_1	Pier shape correction factor		0.90	1.00	0.90	1.00	1.00	1.00	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	
L	Length of pier	m	28.50	28.50	29.49	29.49	28.50	28.50	6.40	7.47	8.53	Ignores empty space between pier columns
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	-0.11	0.78	1.73	1.16	0.75	0.09	-0.76	-1.61	-1.82	z_w - zb
K_2	Correction factor for angle of attack		1.10	2.34	1.28	2.31	2.11	3.38	2.76	3.10	2.30	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		--	0.18	0.45	0.58	0.09	0.13	--	--	--	$V_1/(g*y_1)^{.5}$
ys	Pier scour	m	--	2.32	2.26	4.41	1.59	1.42	--	--	--	$y_1*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 25-YEAR RIVERINE FLOOD DURING CONSTRUCTION STAGE 2

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 28, 2003
Checked by	K. Brennan
Date	26-Jun-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Notes
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.000**	**26.750**	**30.500**	**30.500**	**26.875**	**50.625**	**51.000**	**51.000**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	1.18	1.07	-0.46	-0.47	0.98	1.24	1.18	1.19	1.19	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.07	0.42	1.29	1.45	0.33	0.08	0.07	0.06	0.04	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	2.55	14.73	2.56	12.16	4.53	25.89	39.91	48.96	30.46	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	7.50	7.50	7.50	7.50	7.50	7.50	1.07	1.07	1.07	
L	Length of pier	m	18.00	18.00	18.00	18.00	18.00	18.00	6.40	7.47	8.53	Ignores empty space between pier columns; includes cofferdams
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	-0.16	0.80	1.64	0.99	0.91	0.06	-0.81	-1.65	-1.87	z_w - zb
K_2	Correction factor for angle of attack		1.07	1.34	1.07	1.29	1.12	1.54	2.70	3.18	2.82	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		--	0.15	0.32	0.47	0.11	0.11	--	--	--	$V_1/(g*y_1)^{.5}$
ys	Pier scour	m	--	4.93	7.00	8.31	3.78	1.95	--	--	--	$y_1*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 25-YEAR RIVERINE FLOOD DURING CONSTRUCTION STAGE 3

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 21, 2003
Checked by	K. Brennan
Date	26-Jun-03

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Notes
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.000**	**39.750**	**42.750**	**42.125**	**39.625**	**38.125**	**51.000**	**51.000**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	1.18	0.81	-0.43	-0.23	0.84	1.25	1.18	1.19	1.19	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.06	0.35	1.28	1.28	0.23	0.09	0.07	0.06	0.05	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	10.75	9.35	1.47	11.19	0.62	19.70	39.74	48.74	30.41	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	7.50	7.50	7.50	7.50	7.50	7.50	1.07	1.07	1.07	
L	Length of pier	m	14.50	14.50	14.50	14.50	14.50	14.50	3.20	3.73	4.27	Ignores empty space between pier columns; includes cofferdams
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	-0.16	0.54	1.67	1.22	0.77	0.07	-0.81	-1.66	-1.87	z_w - zb
K_2	Correction factor for angle of attack		1.21	1.19	1.03	1.22	1.01	1.35	1.90	2.17	1.99	$K_2 = (\cos \alpha + L/a \sin \alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		--	0.15	0.32	0.37	0.08	0.11	--	--	--	$V_1/(g^*y_1)^{.5}$
ys	Pier scour	m	--	3.79	6.75	7.62	2.87	1.80	--	--	--	$y_1{*}2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 100-YEAR TIDAL STORM SURGE FLOOD

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 21, 2003
Checked by	K. Brennan
Date	28-May-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	**Pier 6**	**Pier 7**	**Pier 8**	**Pier 9**	**Pier 10**	**Notes**
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.000**	**51.375**	**53.875**	**67.250**	**50.750**	**51.000**	**50.875**	**50.750**	**50.500**	
Hydraulic Data												
z_w	Water Surface Elevation	m	2.76	2.45	0.62	-0.26	2.95	2.75	2.86	2.97	3.14	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.25	0.53	1.17	1.16	0.36	0.28	0.17	0.13	0.06	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	16.13	16.51	1.42	11.93	6.98	22.40	31.60	42.66	17.80	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.00	1.00	0.90	1.00	1.00	1.00	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	
L	Length of pier	m	28.50	28.50	29.49	29.49	28.50	28.50	6.40	7.47	8.53	Ignores empty space between pier columns
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	1.42	2.18	2.72	1.19	2.88	1.57	0.87	0.13	0.08	z_w - zb
K_2	Correction factor for angle of attack		2.58	2.61	1.18	2.24	1.79	3.03	2.46	3.02	2.21	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		0.07	0.11	0.23	0.34	0.07	0.07	0.06	0.11	0.07	$V_1/(g^*y_1)^{.5}$
ys	Pier scour	m	2.07	3.09	1.83	3.43	1.87	2.61	1.74	1.45	0.71	$y_1{}^*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 500-YEAR TIDAL STORM SURGE

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 21, 2003
Checked by	K. Brennan
Date	28-May-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	**Pier 6**	**Pier 7**	**Pier 8**	**Pier 9**	**Pier 10**	**Notes**
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**51.125**	**51.625**	**54.125**	**67.375**	**50.750**	**51.000**	**50.875**	**50.750**	**50.750**	
Hydraulic Data												
z_w	Water Surface Elevation	m	3.06	2.60	0.64	-0.27	3.41	3.17	3.30	3.42	3.43	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.33	0.63	1.24	1.16	0.39	0.34	0.23	0.16	0.12	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	14.93	16.05	1.47	11.88	9.33	23.31	30.46	39.63	18.20	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.00	1.00	0.90	1.00	1.00	1.00	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.07	
L	Length of pier	m	28.50	28.50	29.49	29.49	28.50	28.50	6.40	7.47	8.53	Ignores empty space between pier columns
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	1.72	2.33	2.74	1.19	3.34	1.99	1.31	0.58	0.37	z_w - zb
K_2	Correction factor for angle of attack		2.49	2.57	1.19	2.24	2.01	3.09	2.42	2.93	2.24	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		0.08	0.13	0.24	0.34	0.07	0.08	0.06	0.07	0.06	$V_1/(g^*y_1)^{.5}$
ys	Pier scour	m	2.32	3.33	1.89	3.42	2.22	3.00	2.07	1.91	1.19	$y_1{}^*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 25-YEAR TIDAL STORM SURGE DURING CONSTRUCTION STAGE 2

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 28, 2003
Checked by	K. Brennan
Date	26-Jun-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Notes
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**50.875**	**51.500**	**53.625**	**54.625**	**51.125**	**51.000**	**50.750**	**50.250**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	2.55	2.11	0.67	0.12	2.36	2.46	2.63	2.85	2.47	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.19	0.45	1.09	1.18	0.44	0.24	0.12	0.02	0.07	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	16.99	16.56	1.99	10.50	2.80	20.05	31.72	47.25	21.87	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	7.50	7.50	7.50	7.50	7.50	7.50	1.07	1.07	1.07	
L	Length of pier	m	18.00	18.00	18.00	18.00	18.00	18.00	6.40	7.47	8.53	Ignores empty space between pier columns; includes cofferdams
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	1.21	1.84	2.77	1.57	2.29	1.28	0.64	0.01	-0.59	z_w - zb
K_2	Correction factor for angle of attack		1.39	1.38	1.05	1.26	1.07	1.45	2.46	3.14	2.43	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		0.05	0.11	0.21	0.30	0.09	0.07	0.05	0.10	--	$V_1/(g*y_1)^{.5}$
ys	Pier scour	m	3.79	5.84	6.87	7.87	4.62	4.40	1.46	0.49	--	$y_1*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

EXISTING CONDITIONS: 25-YEAR TIDAL STORM SURGE DURING CONSTRUCTION STAGE 3

PIER SCOUR COMPUTATIONS FOR UNIFORM PIER

Made by	J. Sampson
Date	May 28, 2003
Checked by	K. Brennan
Date	26-Jun-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	**Pier 6**	**Pier 7**	**Pier 8**	**Pier 9**	**Pier 10**	**Notes**
USER DATA REQUIRED FOR CALCULATION												
Time	51.000		**50.875**	**51.375**	**53.625**	**54.625**	**51.000**	**51.000**	**50.750**	**50.250**	**51.000**	
Hydraulic Data												
z_w	Water Surface Elevation	m	2.55	2.20	0.67	0.13	2.45	2.46	2.63	2.85	2.47	
z_b	Bottom elevation at start of computations	m	1.34	0.27	-2.10	-1.45	0.07	1.18	1.99	2.84	3.06	
V_1	Approach velocity at start of computations	m/sec	0.18	0.42	1.08	1.16	0.35	0.25	0.13	0.02	0.08	
θ	Angle of Pier	Degrees	94.08	94.05	105.59	105.59	94.05	94.05	94.09	94.09	66.02	
α	Angle of Attack	Degrees	15.27	11.92	2.10	10.04	4.69	16.21	31.66	47.42	21.24	
Bed Conditions												
K_3	Correction for Bed Forms		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	
K_4	Bed Armoring		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	K_4 = 1: Bed sediment D_{90} << 20 mm
Pier Data												
K_1	Pier shape correction factor		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	See HEC-18 Table 6.1
a	Pier width	m	7.50	7.50	7.50	7.50	7.50	7.50	1.07	1.07	1.07	
L	Length of pier	m	14.50	14.50	14.50	14.50	14.50	14.50	3.20	3.73	4.27	Ignores empty space between pier columns; includes cofferdams
FOLLOWING ARE CALCULATIONS												
y_1	Approach flow depth at the beginning of computations	m	1.21	1.93	2.77	1.58	2.38	1.28	0.64	0.01	-0.59	z_w - zb
K_2	Correction factor for angle of attack		1.29	1.23	1.05	1.20	1.10	1.30	1.78	2.15	1.76	$K_2 = (\cos\alpha + L/a \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
Fr	Froude Number		0.05	0.10	0.21	0.30	0.07	0.07	0.05	0.10	--	$V_1/(g*y_1)^{.5}$
ys	Pier scour	m	3.47	5.08	6.79	7.46	4.31	4.07	1.06	0.34	--	$y_1*2K_1K_2K_3K_4(a/y_1)^{0.65}(Fr)^{.43}$

I-95 OVER WEST RIVER

Proposed Conditions: 100-YEAR RIVERINE FLOOD

PIER SCOUR COMPUTATIONS FOR CASE II SCOUR

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**25.125**	**41.5**	**44.5**	**25.125**	**25.125**	
z_w	Water Surface Elevation	m	1.30	0.04	-0.52	1.29	1.30	NGVD
V_1	Approach velocity at start of computations	m/sec	0.05	1.24	0.57	0.09	0.05	
α	Angle of Flow	Degrees	-110.81	-72.14	-78.38	-119.75	-128.69	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.00	0.11	0.09	0.00	0.00	$6.19*y_1\wedge^{1/6}*d_{50}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.00	0.04	0.03	0.00	0.00	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.00	0.23	0.18	0.00	0.00	$6.19*y_1\wedge^{1/6}*d_{95}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.00	0.09	0.07	0.00	0.00	HEC-18 Eq. 6.7
V_r			0.00	45.80	26.76	0.00	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	24.30	13.77	7.53	33.84	17.77	
K_{2S}	Correction Factor for Angle of Attack		2.21	2.39	1.84	2.75	2.08	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075-.0669\, f/a_{pier})-(.4271-.0778\, f/a_{pier})\, h_1/a_{pier}+(.1615-.0455\, f/a_{pier})\,(h_1/a_{pier})^2-(.0269-.012\, f/a_{pier})\,(h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		1.0	1.0	1.0	1.0	1.0	
Y_1	Flow Depth Upstream of Pier	m	0.00	0.15	0.03	0.00	0.00	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0000	1.0089	1.0282	0.0000	0.0000	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.00	3.41	1.29	0.00	0.00	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.49	-1.98	-2.60	-2.59	-2.07	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.00	1.86	0.68	0.00	0.00	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.00	0.10	0.03	0.00	0.00	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	0.60	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.00	1.86	0.68	0.00	0.00	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	0.00	6.60	6.60	0.00	0.00	$a_{pc}[\exp\{-2.705+.0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.00	0.02	0.01	0.00	0.00	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.70	1.85	1.52	3.33	2.61	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.00	0.15	0.06	0.00	0.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.00	-0.03	0.00	0.00	
y_f	distance from the bed to the top of footing	m	-1.99	0.02	-0.60	-1.09	-0.57	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.05	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.00	3.46	1.29	0.00	0.00	$ys_{pier} + ys_{pc} + ys_{pg}$

Made by	J. Sampson
Date	18-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

I-95 OVER WEST RIVER

Proposed Conditions: 25-YEAR RIVERINE FLOOD

PIER SCOUR COMPUTATIONS FOR CASE II SCOUR

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**25.125**	**41.500**	**32.500**	**25.125**	**25.125**	
z_w	Water Surface Elevation	m	1.28	-0.02	-0.54	1.27	1.28	NGVD
V_1	Approach velocity at start of computations	m/sec	0.04	1.06	0.42	0.07	0.04	
α	Angle of Flow	Degrees	-106.57	-72.36	-77.01	-120.03	-128.69	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.00	0.11	0.08	0.00	0.00	$6.19*y_1\wedge^{1/6}*d_{50}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.00	0.04	0.03	0.00	0.00	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.00	0.21	0.15	0.00	0.00	$6.19*y_1\wedge^{1/6}*d_{95}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.00	0.08	0.06	0.00	0.00	HEC-18 Eq. 6.7
V_r			0.00	41.97	22.45	0.00	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	20.06	13.55	8.91	34.12	17.77	
K_{2S}	Correction Factor for Angle of Attack		2.04	2.37	1.97	2.75	2.08	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		1.0	1.0	1.0	1.0	1.0	
Y_1	Flow Depth Upstream of Pier	m	0.00	0.10	0.01	0.00	0.00	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0000	1.0815	1.1360	0.0000	0.0000	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.00	2.96	1.09	0.00	0.00	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.49	-2.20	-2.70	-2.59	-2.07	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.00	1.58	0.56	0.00	0.00	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.00	0.06	0.01	0.00	0.00	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	0.58	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.00	1.58	0.56	0.00	0.00	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	0.00	6.60	6.60	0.00	0.00	$a_{pc}[\exp\{-2.705+.0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.00	0.02	0.00	0.00	0.00	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.47	1.84	1.61	3.34	2.61	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc}\sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.00	0.11	0.03	0.00	0.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	-0.01	-0.01	0.00	0.00	
y_f	distance from the bed to the top of footing	m	-1.99	-0.20	-0.70	-1.09	-0.57	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.00	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.00	2.96	1.09	0.00	0.00	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 500-YEAR RIVERINE FLOOD
PIER SCOUR COMPUTATIONS FOR CASE II SCOUR

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

SI Units								
(see HEC-18, 4th Edition, Section 6.2)			**Pier 1**	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	
	Description	Units						**Notes**
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**25.125**	**41.625**	**27.125**	**25.125**	**25.125**	
z_w	Water Surface Elevation	m	1.32	0.07	0.92	1.32	1.33	NGVD
V_1	Approach velocity at start of computations	m/sec	0.06	1.46	0.69	0.11	0.06	
α	Angle of Flow	Degrees	-115.22	-72.02	-78.72	-119.18	-128.63	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.00	0.12	0.17	0.00	0.00	$6.19*y_1$^$^{1/6}*d_{50}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.00	0.04	0.06	0.00	0.00	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.00	0.24	0.33	0.00	0.00	$6.19*y_1$^$^{1/6}*d_{95}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.00	0.09	0.13	0.00	0.00	HEC-18 Eq. 6.7
V_r			0.00	52.64	16.52	0.00	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	28.71	13.89	7.19	33.26	17.72	
K_{2S}	Correction Factor for Angle of Attack		2.37	2.40	1.81	2.73	2.08	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		1.0	1.0	0.6	1.0	1.0	
Y_1	Flow Depth Upstream of Pier	m	0.00	0.18	1.47	0.00	0.00	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0000	1.0882	0.1809	0.0000	0.0000	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.00	3.76	1.42	0.00	0.00	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.49	-1.80	-2.54	-2.59	-2.07	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.00	2.07	2.19	0.00	0.00	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.00	0.13	0.46	0.00	0.00	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	0.63	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.00	2.07	2.19	0.00	0.00	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	0.00	6.60	6.60	0.00	0.00	a_{pc} [exp{-2.705 +.0.51 ln(T*/y_{2^*}) - 2.783$(h_2/y_{2^*})^3$ + 1.751/exp(h_2/y_{2^*})}]
Fr_{pc}	Pile Cap Froude Number		0.00	0.03	0.10	0.00	0.00	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.92	1.86	1.50	3.30	2.60	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.00	0.35	0.49	0.00	0.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.01	-0.14	0.00	0.00	
y_f	distance from the bed to the top of footing	m	-1.99	0.20	-0.54	-1.09	-0.57	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.43	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.00	4.19	1.42	0.00	0.00	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 25-YEAR RIVERINE FLOOD DURING STAGE 2 CONSTRUCTION
PIER SCOUR COMPUTATIONS FOR UNIFORM PIERS 1-4, AND CASE II PIER 5

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**51.000**	**41.625**	**43.250**	**51.000**	**51.000**	
z_w	Water Surface Elevation	m	1.18	1.18	1.17	1.18	1.19	NGVD
V_1	Approach velocity at start of computations	m/sec	0.05	0.47	0.37	0.08	0.05	
α	Angle of Flow	Degrees	-84.40	-73.63	-76.95	-122.56	-156.36	
Pier Data								
	Cofferdam or Pier							
a	Pier width	m	6.00	9.00	9.00	6.00	1.52	
L	Length of pier	m	28.80	30.30	32.20	33.00	7.62	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.1	1.1	1.1	1.1	1.0	
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-999	-999	-999	-999	-2.07	=-999 for no pile cap (as is the case for the cofferdams)
T	Thickness of pile cap	m	1.5	1.5	1.5	1.5	1.5	
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	37.35	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	2.03	2.03	1.22	12.99	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles	m	3	5	5	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.00	0.16	0.17	0.00	0.00	$6.19*y_1{}^{\wedge 1/6}*d_{50}{}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.00	0.05	0.05	0.00	0.00	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.00	0.33	0.34	0.00	0.00	$6.19*y_1{}^{\wedge 1/6}*d_{95}{}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.00	0.12	0.12	0.00	0.00	HEC-18 Eq. 6.7
V_r			0.00	9.09	6.51	0.00	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		4.80	3.37	3.58	5.50	5.00	
α	Angle of Attack	Degrees	2.11	12.29	8.96	36.64	45.45	
K_{2S}	Correction Factor for Angle of Attack		1.11	1.41	1.33	2.50	2.57	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.25	0.17	0.17	0.25	0.98	
h_1/a_{pier}			-166.25	-110.83	-110.83	-166.25	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.00	1.00	1.00	1.00	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		1.0	0.5	0.4	1.0	1.0	
Y_1	Flow Depth Upstream of Pier	m	0.00	1.30	1.73	0.00	0.00	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-997.50	-997.50	-997.50	-997.50	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0000	0.1315	0.0894	0.0000	0.0000	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.00	3.05	2.53	0.00	0.00	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-999.00	-997.48	-997.73	-999.00	-2.07	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.00	2.82	2.99	0.00	0.00	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.00	0.22	0.21	0.00	0.00	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	1.50	1.50	1.50	0.49	Check for Top of Pile Cap Above Water Surface
$y_2{}^*$	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.00	2.82	2.99	0.00	0.00	Max value of y_2 is 3.5 a_{pc}
a_{pc^*}	Equivalent full depth solid pier width	m	0.00	6.60	6.60	0.00	0.00	$a_{pc}[\exp\{-2.705+.0.51\ \ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.00	0.04	0.04	0.00	0.00	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		1.20	1.78	1.61	3.45	3.29	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.00	0.40	0.40	0.00	0.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.00	0.00	0.00	0.00	
y_f	distance from the bed to the top of footing	m	-997.50	-995.98	-996.23	-997.50	-0.57	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.00	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f{}^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.00	3.05	2.53	0.00	0.00	$ys_{pier}+ys_{pc}+ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 25-YEAR RIVERINE FLOOD DURING STAGE 3 CONSTRUCTION
PIER SCOUR COMPUTATIONS FOR UNIFORM PIERS 1-4, AND CASE II PIER 5

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**51.000**	**41.625**	**30.875**	**51.000**	**51.000**	
z_w	Water Surface Elevation	m	1.18	-0.04	-0.53	1.18	1.19	NGVD
V_1	Approach velocity at start of computations	m/sec	0.04	0.90	0.53	0.08	0.05	
α	Angle of Flow	Degrees	-66.98	-75.06	-79.70	-120.78	-144.39	
Pier Data								
	Cofferdam or Pier							
a	Pier width	m	6.00	9.00	9.00	6.00	1.52	
L	Length of pier	m	44.40	47.10	48.60	48.90	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.1	1.1	1.1	1.1	1.0	
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-999	-999	-999	-999	-2.07	=-999 for no pile cap (as is the case for the cofferdams)
T	Thickness of pile cap	m	1.5	1.5	1.5	1.5	1.5	
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	2.03	2.03	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles	m	3	5	5	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.00	0.10	0.08	0.00	0.00	$6.19*y_1{}^{\wedge 1/6}*d_{50}{}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.00	0.03	0.03	0.00	0.00	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.00	0.21	0.17	0.00	0.00	$6.19*y_1{}^{\wedge 1/6}*d_{95}{}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.00	0.07	0.06	0.00	0.00	HEC-18 Eq. 6.7
V_r			0.00	30.20	21.30	0.00	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		7.40	5.23	5.40	8.15	7.00	
α	Angle of Attack	Degrees	19.53	10.85	6.21	34.87	33.47	
K_{2S}	Correction Factor for Angle of Attack		2.22	1.55	1.35	3.02	2.73	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.25	0.17	0.17	0.25	0.98	
h_1/a_{pier}			-166.25	-110.83	-110.83	-166.25	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.00	1.00	1.00	1.00	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		1.0	1.0	1.0	1.0	1.0	
Y_1	Flow Depth Upstream of Pier	m	0.00	0.08	0.03	0.00	0.00	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-997.50	-997.50	-997.50	-997.50	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0000	1.0246	1.0763	0.0000	0.0000	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.00	6.51	3.85	0.00	0.00	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-999.00	-995.74	-997.07	-999.00	-2.07	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.00	3.34	1.95	0.00	0.00	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.00	0.02	0.01	0.00	0.00	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	1.50	1.50	1.50	0.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.00	3.34	1.95	0.00	0.00	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	0.00	6.60	6.60	0.00	0.00	$a_{pc}[\exp\{-2.705+.0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.00	0.00	0.00	0.00	0.00	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.44	1.70	1.44	3.38	3.54	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc}\sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.00	0.05	0.03	0.00	0.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.00	0.00	0.00	0.00	
y_f	distance from the bed to the top of footing	m	-997.50	-994.24	-995.57	-997.50	-0.57	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.00	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f{}^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.00	6.51	3.85	0.00	0.00	$ys_{pier}+ys_{pc}+ys_{pg}$

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

I-95 OVER WEST RIVER

Proposed Conditions: 100-YEAR TIDAL STORM SURGE FLOOD

PIER SCOUR COMPUTATIONS FOR CASE II SCOUR

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**50.750**	**52.000**	**52.750**	**50.625**	**50.750**	
z_w	Water Surface Elevation	m	2.96	1.95	1.38	3.05	2.97	NGVD
V_1	Approach velocity at start of computations	m/sec	0.15	0.90	0.74	0.19	0.09	
α	Angle of Flow	Degrees	-69.67	-70.19	-79.87	-112.59	-130.92	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.13	0.18	0.17	0.16	0.12	$6.19 * y_1{}^{\wedge 1/6} * d_{50}{}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.04	0.06	0.06	0.06	0.04	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.25	0.35	0.35	0.33	0.24	$6.19 * y_1{}^{\wedge 1/6} * d_{95}{}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.10	0.14	0.13	0.13	0.09	HEC-18 Eq. 6.7
V_r			4.05	20.65	17.08	3.74	1.79	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	16.84	15.72	6.04	26.67	20.01	
K_{2S}	Correction Factor for Angle of Attack		1.90	2.55	1.70	2.48	2.19	$K_{2S} = (\cos \alpha + L_s / a_s \sin \alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075 - .0669\ f/a_{pier}) - (.4271 - .0778\ f/a_{pier})\ h_1/a_{pier} + (.1615 - .0455\ f/a_{pier})\ (h_1/a_{pier})^2 - (.0269 - .012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		0.4	1.0	1.0	1.0	0.4	
Y_1	Flow Depth Upstream of Pier	m	0.27	2.07	1.93	1.27	0.20	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0930	0.1988	0.1706	0.0525	0.0620	$V_1/(g * y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.57	4.48	2.33	1.44	0.20	$y_1 * K_{hpier} * 2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.21	-1.44	-2.08	-1.87	-1.96	$h_0 + ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.55	4.30	3.10	1.98	0.30	$y_1 + ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.07	0.43	0.46	0.12	0.06	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	1.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.55	4.30	3.10	1.98	0.30	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	4.80	6.60	6.60	4.80	4.80	$a_{pc}[\exp\{-2.705 + .0.51 \ln(T^*/y_{2^*}) - 2.783(h_2/y_{2^*})^3 + 1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.03	0.07	0.08	0.03	0.03	$V_2/(g * y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.28	1.95	1.43	2.98	2.76	$K_{2pc} = (\cos \alpha + L_{pc} / a_{pc} \sin \alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.13	0.48	0.49	0.18	0.11	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		-0.27	0.07	-0.01	-0.03	-0.09	
y_f	distance from the bed to the top of footing	m	-1.71	0.56	-0.08	-0.37	-0.46	$h_0 + T + ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	1.38	0.00	0.00	0.00	$y_f * 2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f{}^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.57	5.86	2.33	1.44	0.20	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER			*Made by*	J. Sampson
Proposed Conditions: 25-YEAR TIDAL STORM SURGE FLOOD			*Date*	20-Aug-03
PIER SCOUR COMPUTATIONS FOR CASE II SCOUR			*Checked by*	K. Brennan
			Date	22-Aug-03

SI Units								
(see HEC-18, 4th Edition, Section 6.2)			**Pier 1**	**Pier 2**	**Pier 3**	**Pier 4**	**Pier 5**	
	Description	Units						**Notes**
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**50.500**	**51.875**	**52.500**	**50.875**	**50.500**	
z_w	Water Surface Elevation	m	2.78	1.81	1.37	2.53	2.78	NGVD
V_1	Approach velocity at start of computations	m/sec	0.09	0.81	0.68	0.19	0.05	
α	Angle of Flow	Degrees	-68.41	-70.24	-79.98	-111.90	-132.30	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.10	0.17	0.17	0.15	0.07	$6.19*y_1\wedge^{1/6}*d_{50}{}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.04	0.06	0.06	0.05	0.03	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.21	0.35	0.35	0.30	0.15	$6.19*y_1\wedge^{1/6}*d_{95}{}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.08	0.13	0.13	0.12	0.06	HEC-18 Eq. 6.7
V_r			2.48	18.65	15.58	4.34	1.58	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	18.10	15.67	5.93	25.99	21.38	
K_{2S}	Correction Factor for Angle of Attack		1.96	2.54	1.69	2.45	2.25	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075-.0669\, f/a_{pier})-(.4271-.0778\, f/a_{pier})\, h_1/a_{pier}+(.1615-.0455\, f/a_{pier})\,(h_1/a_{pier})^2-(.0269-.012\, f/a_{pier})\,(h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		0.4	1.0	1.0	0.5	0.3	
Y_1	Flow Depth Upstream of Pier	m	0.08	1.93	1.93	0.75	0.01	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0997	0.1849	0.1569	0.0695	0.1486	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.35	4.23	2.23	0.62	0.10	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.32	-1.56	-2.13	-2.28	-2.02	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.26	4.05	3.04	1.06	0.06	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.03	0.38	0.43	0.13	0.01	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	1.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.26	4.05	3.04	1.06	0.06	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	4.80	6.60	6.60	4.80	0.00	$a_{pc}[\exp\{-2.705+.0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.02	0.06	0.08	0.04	0.01	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.36	1.95	1.42	2.94	2.85	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.07	0.47	0.48	0.19	1.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		-0.24	0.05	-0.02	-0.11	-0.09	
y_f	distance from the bed to the top of footing	m	-1.82	0.44	-0.13	-0.78	-0.52	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	1.14	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.35	5.37	2.23	0.62	0.10	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 500-YEAR TIDAL STORM SURGE FLOOD
PIER SCOUR COMPUTATIONS FOR CASE II SCOUR

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	23-Aug-03

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**50.875**	**52.125**	**53.000**	**50.875**	**50.750**	
z_w	Water Surface Elevation	m	3.30	2.15	1.43	3.30	3.43	NGVD
V_1	Approach velocity at start of computations	m/sec	0.22	1.02	0.83	0.30	0.13	
α	Angle of Flow	Degrees	-70.70	-70.13	-79.93	-112.64	-130.06	
Pier Data								
	Pier Stem							
a	Pier width	m	1.52	2.40	2.40	1.52	1.52	
L	Length of pier	m	9.14	42.20	44.70	10.67	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.0	1.0	1.0	1.0	1.0	Circular Columns or Round Nose Plinths
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-3.49	-3.68	-3.25	-2.59	-2.07	
T	Thickness of pile cap	m	1.5	2	2	1.5	1.5	Takes into account sheet piling at Piers 2 and 3
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	1.22	1.22	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	3	3	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.14	0.18	0.17	0.17	0.15	$6.19*y_1$^$^{1/6}*d_{50}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.05	0.06	0.06	0.06	0.05	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.29	0.36	0.35	0.34	0.29	$6.19*y_1$^$^{1/6}*d_{95}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.11	0.14	0.13	0.13	0.11	HEC-18 Eq. 6.7
V_r			5.70	23.35	19.09	6.91	2.51	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		6.00	17.58	18.63	7.00	7.00	
α	Angle of Attack	Degrees	15.81	15.78	5.98	26.73	19.15	
K_{2S}	Correction Factor for Angle of Attack		1.86	2.55	1.69	2.48	2.15	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.98	0.63	0.63	0.98	0.98	
h_1/a_{pier}			-1.31	-0.70	-0.52	-0.71	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.03	0.70	0.60	0.66	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})(h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})(h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		0.5	1.0	1.0	1.0	0.4	
Y_1	Flow Depth Upstream of Pier	m	0.61	2.27	1.98	1.51	0.66	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-1.99	-1.68	-1.25	-1.09	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0914	0.2161	0.1874	0.0785	0.0503	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.81	4.81	2.44	1.82	0.31	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-3.09	-1.28	-2.03	-1.68	-1.91	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	1.01	4.67	3.20	2.42	0.81	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.13	0.50	0.51	0.19	0.10	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	2.00	2.00	1.50	1.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	1.01	4.67	3.20	2.42	0.81	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	4.80	6.60	6.60	4.80	4.80	$a_{pc}[\exp\{-2.705+.0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.04	0.07	0.09	0.04	0.04	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.22	1.95	1.43	2.98	2.70	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.19	0.50	0.50	0.41	0.16	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		-0.25	0.10	0.00	-0.02	-0.06	
y_f	distance from the bed to the top of footing	m	-1.59	0.72	-0.03	-0.18	-0.41	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	1.71	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.81	6.52	2.44	1.82	0.31	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 25-YEAR TIDAL STORM SURGE DURING STAGE 2 CONSTRUCTION

PIER SCOUR COMPUTATIONS FOR UNIFORM PIERS 1-4, AND CASE II PIER 5

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	23-Aug-03

SI Units

(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**50.500**	**55.000**	**52.625**	**50.875**	**50.500**	
z_w	Water Surface Elevation	m	2.78	2.78	2.77	2.78	2.78	NGVD
V_1	Approach velocity at start of computations	m/sec	0.09	0.24	0.22	0.12	0.05	
α	Angle of Flow	Degrees	-68.18	-73.80	-78.38	-109.84	-131.78	
Pier Data								
	Cofferdam or Pier							
a	Pier width	m	6.00	9.00	9.00	6.00	1.52	
L	Length of pier	m	28.80	30.30	32.20	33.00	7.62	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.1	1.1	1.1	1.1	1.0	
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-999	-999	-999	-999	-2.07	=-999 for no pile cap (as is the case for the cofferdams)
T	Thickness of pile cap	m	1.5	1.5	1.5	1.5	1.5	
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	37.35	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	2.03	2.03	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	5	5	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.10	0.19	0.19	0.16	0.08	$6.19*y_1\wedge^{1/6}*d_{50}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.03	0.06	0.06	0.05	0.03	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.21	0.37	0.38	0.31	0.15	$6.19*y_1\wedge^{1/6}*d_{95}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.08	0.13	0.14	0.11	0.06	HEC-18 Eq. 6.7
V_r			2.08	3.35	3.00	1.72	1.55	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		4.80	3.37	3.58	5.50	5.00	
α	Angle of Attack	Degrees	18.33	12.11	7.53	23.92	20.87	
K_{2S}	Correction Factor for Angle of Attack		1.79	1.40	1.28	2.11	1.91	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.25	0.17	0.17	0.25	0.98	
h_1/a_{pier}			-166.25	-110.83	-110.83	-166.25	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.00	1.00	1.00	1.00	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		0.3	0.4	0.4	0.4	0.3	
Y_1	Flow Depth Upstream of Pier	m	0.09	2.89	3.33	0.99	0.01	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-997.50	-997.50	-997.50	-997.50	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0997	0.0441	0.0389	0.0392	0.1403	$V_1/(g*y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.70	2.12	1.90	1.42	0.09	$y_1*K_{hpier}*2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-998.65	-997.94	-998.05	-998.29	-2.03	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.44	3.96	4.28	1.70	0.06	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.02	0.17	0.17	0.07	0.01	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	1.50	1.50	1.50	1.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.44	3.96	4.28	1.70	0.06	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	4.80	6.60	6.60	4.80	0.00	$a_{pc}[\exp(-2.705+0.51\ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*}))]$
Fr_{pc}	Pile Cap Froude Number		0.01	0.03	0.03	0.02	0.02	$V_2/(g*y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.37	1.77	1.52	2.83	2.34	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.05	0.21	0.21	0.13	1.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.00	0.00	0.00	-0.12	
y_f	distance from the bed to the top of footing	m	-997.15	-996.44	-996.55	-996.79	-0.53	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.00	0.00	0.00	0.00	$y_f*2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.70	2.12	1.90	1.42	0.09	$ys_{pier} + ys_{pc} + ys_{pg}$

I-95 OVER WEST RIVER

Proposed Conditions: 25-YEAR TIDAL STORM SURGE DURING STAGE 3 CONSTRUCTION
PIER SCOUR COMPUTATIONS FOR UNIFORM PIERS 1-4, AND CASE II PIER 5

Made by	J. Sampson
Date	20-Aug-03
Checked by	K. Brennan
Date	22-Aug-03

SI Units
(see HEC-18, 4th Edition, Section 6.2)

	Description	Units	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Notes
USER DATA REQUIRED FOR CALCULATION								
Simulation Data								
Time	51.000		**49.750**	**55.125**	**52.625**	**50.875**	**50.500**	
z_w	Water Surface Elevation	m	2.80	2.80	2.80	2.80	2.80	NGVD
V_1	Approach velocity at start of computations	m/sec	0.08	0.28	0.30	0.10	0.02	
α	Angle of Flow	Degrees	111.55	-75.34	-78.67	-109.49	-133.61	
Pier Data								
	Cofferdam or Pier							
a	Pier width	m	6.00	9.00	9.00	6.00	1.52	
L	Length of pier	m	44.40	47.10	48.60	48.90	10.67	
z_b	Bottom elevation at start of computations	m	2.69	-0.12	-0.55	1.79	2.77	NGVD
θ	Angle of Pier	Degrees	93.49	94.09	94.09	94.09	69.09	
K_1	Pier shape correction factor		1.1	1.1	1.1	1.1	1.0	
f	Distance between front edge of pile cap or footing and pier		1.5	1.5	1.5	1.5	1.5	
	Pile Cap							
h_0	Height of pile cap above bed at start	m	-999	-999	-999	-999	-2.07	=-999 for no pile cap (as is the case for the cofferdams)
T	Thickness of pile cap	m	1.5	1.5	1.5	1.5	1.5	
K_{1pc}	Pile Cap shape correction factor		1.1	1.1	1.1	1.1	1.1	Square Pile Cap - See HEC-18 Table 6.1
a_{pc}	Pile cap width	m	4.8	6.6	6.6	4.8	4.8	
L_{pc}	Length of pile cap	m	43.2	44.7	46.2	47.7	53.7	
	Pile Group							
K_{1pg}	Pile Group shape correction factor		1	1	1	1	1	Circular Piles
S	Center to Center spacing of piles	m	1.5	1.5	1.5	1.5	1.5	
a_{proj}	Projected composite width of pile group	m	1.22	2.03	2.03	1.22	1.22	
a	Diameter of single pile	m	0.406	0.406	0.406	0.406	0.406	
m	Number of rows of aligned piles		3	5	5	3	3	
Bed Conditions								
D_{50}	Median Grain Diameter	mm	0.016	0.016	0.016	0.016	0.016	
D_{84}	Grain roughness or 84% grain size	mm	0.08	0.08	0.08	0.08	0.08	
D_{95}	95% Grain size	mm	0.13	0.13	0.13	0.13	0.13	
D_{50}	Median Grain Diameter	feet	0.0001	0.0001	0.0001	0.0001	0.0001	
D_{84}	Grain roughness or 84% grain size	feet	0.0002	0.0002	0.0002	0.0002	0.0002	
D_{95}	95% Grain size	feet	0.0004	0.0004	0.0004	0.0004	0.0004	
V_cD_{50}	Critical Velocity for D_{50}	m/s	0.11	0.19	0.19	0.16	0.09	$6.19{*}y_1{}^{\wedge 1/6}{*}d_{50}{}^{1/3}$
$V_{ic}D_{50}$	Velocity required to initiate scour for D_{50}	m/s	0.04	0.06	0.06	0.05	0.03	HEC-18 Eq. 6.7
V_cD_{95}	Critical Velocity for D_{95}	m/s	0.22	0.37	0.38	0.31	0.18	$6.19{*}y_1{}^{\wedge 1/6}{*}d_{95}{}^{1/3}$
$V_{ic}D_{95}$	Velocity required to initiate scour for D_{95}	m/s	0.08	0.13	0.14	0.11	0.07	HEC-18 Eq. 6.7
V_r			1.57	4.14	4.48	1.06	0.00	HEC-18 Eq. 6.6
Scour Calculations								
	Pier Stem							
L/a	Length/Width Ratio		7.40	5.23	5.40	8.15	7.00	
α	Angle of Attack	Degrees	18.06	10.57	7.25	23.58	22.70	
K_{2S}	Correction Factor for Angle of Attack		2.15	1.54	1.40	2.53	2.31	$K_{2S} = (\cos\alpha + L_s/a_s \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_3	Bed Form Correction Factor		1.10	1.10	1.10	1.10	1.10	
K_4	Armoring Correction Factor		1.00	1.00	1.00	1.00	1.00	See pg. 6.6 HEC-18
f/a_{pier}			0.25	0.17	0.17	0.25	0.98	
h_1/a_{pier}			-166.25	-110.83	-110.83	-166.25	-0.37	
K_{Hpier}	Coefficient to account for height of pier stem above bed and pile cap overhang f		1.00	1.00	1.00	1.00	0.49	$(.4075-.0669\ f/a_{pier})-(.4271-.0778\ f/a_{pier})\ h_1/a_{pier}+(.1615-.0455\ f/a_{pier})\ (h_1/a_{pier})^2-(.0269-.012\ f/a_{pier})\ (h_1/a_{pier})^3$; =1 for no overhang
K_w	Wide pier adjustment factor		0.3	0.4	0.4	0.3	0.3	
Y_1	Flow Depth Upstream of Pier	m	0.11	2.92	3.36	1.01	0.03	z_w - zb
h_1	Height of pier stem above bed at beginning of computations	m	-997.50	-997.50	-997.50	-997.50	-0.57	$T+h_o$
Fr_{pier}	Froude Number		0.0777	0.0518	0.0526	0.0302	0.0316	$V_1/(g{*}y_1)^{.5}$
ys_{pier}	Scour Depth	m	0.80	2.61	2.56	1.45	0.06	$y_1{*}K_{hpier}{*}2K_{1pier}K_{2pier}K_3K_4(a_{pier}/y_1)^{0.65}(Fr_{pier})^{.43}$
	Pile Cap							
h_2	Adjusted height of pile cap above bed	m	-998.60	-997.70	-997.72	-998.28	-2.04	$h_0+ys_{pier}/2$
y_2	Adjusted flow depth for pile cap scour	m	0.51	4.22	4.63	1.74	0.06	$y_1+ys_{pier}/2$
V_2	Adjusted velocity for pile cap scour	m/s	0.02	0.19	0.22	0.06	0.01	$V_1(y_1/y_2)$
T*	Effective Pile Cap Thickness	m	1.50	1.50	1.50	1.50	1.50	Check for Top of Pile Cap Above Water Surface
y_2*	Effective Flow Depth (for Equivalent full depth solid pier width)	m	0.51	4.22	4.63	1.74	0.06	Max value of y_2 is 3.5 a_{pc}
a_{pc}*	Equivalent full depth solid pier width	m	4.80	6.60	6.60	4.80	0.00	$a_{pc}\ [\exp\{-2.705+.0.51\ \ln(T^*/y_{2^*})-2.783(h_2/y_{2^*})^3+1.751/\exp(h_2/y_{2^*})\}]$
Fr_{pc}	Pile Cap Froude Number		0.01	0.03	0.03	0.01	0.01	$V_2/(g{*}y_2)^{.5}$
K_{2pc}	Correction factor for angle of attack		2.36	1.68	1.50	2.81	2.93	$K_{2pc} = (\cos\alpha + L_{pc}/a_{pc} \sin\alpha)^{0.65}$ HEC-18 Eq. 6.4
K_w	Wide pier adjustment factor		0.05	0.39	0.41	0.11	1.00	See HEC-18 p. 6.7
V_f	Average Velocity in the flow zone below the top of footing		0.00	0.00	0.00	0.00	-0.09	
y_f	distance from the bed to the top of footing	m	-997.10	-996.20	-996.22	-996.78	-0.54	$h_0+T+ys_{pier}/2$
ys_{pc}	Pile Cap Scour	m	0.00	0.00	0.00	0.00	0.00	$y_f{*}2K_{1pc}K_{2pc}K_3K_4K_w(a_{pc}/y_2)^{0.65}(V_f/(g^{1/2}y_f{}^{1/2}))^{.43}$
Total Scour								
Y_s	Scour Depth	m	0.80	2.61	2.56	1.45	0.06	$ys_{pier} + ys_{pc} + ys_{pg}$

APPENDIX D:

SUBSTRUCTURE FOUNDATION COMPUTATIONS